New England Water Works Association
ORGANIZED 1882

VOL. LIX SEPTEMBER, 1945 No. 3

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MILITARY USE OF PUBLIC WATER SUPPLIES

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[*Read May 17, 1945.*]

The relation of the Army to public water supply organizations can be understood clearly if certain facts regarding the military organization are kept in mind. Aside from the direct consumer viewpoint, there are two elements of the Army that are vitally interested in water supplies, namely, the Medical Department and the Corps of Engineers.

The Engineers are charged with the responsibility of providing a satisfactory supply, either by construction of the necessary facilities, or by purchase of water from an existing system. Where water is purchased, it is the Engineers who enter into a contract and establish the specifications that govern the quantity and quality of the service rendered. Furthermore, the Engineers are responsible for the maintenance of military distribution systems and for the operation of treatment devices in such systems.

The Medical Department is charged with the responsibility for determining the sanitary quality of all water supplies and recommending measures to protect the health of military personnel. The function of the Medical Department, in this connection, is similar to that of a public health department. To carry out its mission, the Medical Department maintains an inspection service, and when the occasion demands, makes appropriate recommendations to insure that the water delivered to troops is of satisfactory sanitary quality. The scope of such inspections is practically unlimited so long as any question materially involves the health of troops. It is not enough to consider the military establishment alone, but the inspections must extend to the very source of a supply and must include all the ele-

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ments of storage, treatment, physical condition of structures, and even the qualifications of operating or supervising personnel that may affect the safety of a supply.

The organization of the medical inspection service varies considerably with local conditions. Obviously, situations in foreign theaters require a different approach than in the continental United States, but even in different sections of this country there is considerable divergence in organization and methods. The present discussion will deal mainly with the First Service Command, which is composed of the six New England States.

Water inspection begins basically at each individual installation with a water sampling and testing program. Samples are collected either by a medical officer or qualified enlisted technician and submitted to a laboratory for bateriological examination. Also, at stated intervals, tests are made for residual chlorine at various points of use. These tests are independent of those made by Engineer personnel who are responsible for proper operation of treatment devices.

At several of the larger posts and camps there are assigned Sanitary Corps engineers, who are charged with the duty of making technical inspections, not only at their own stations, but at all surrounding installations within a definite area or district. In this way, competent supervision can be maintained over even the smallest units. These officers are the backbone of the water inspection service, and their judgment and experience decide the frequency of inspection and sampling, the degree of treatment required, and the sanitary precautions to be observed. They must interpret the results of laboratory examinations, either chemical or bacteriological, and must take any needed action on the spot to protect the health of troops. Without exception, these men are qualified engineers, many of them with years of experience in state health department work.

The First Service Command is responsible for all water sanitation at Army installations in New England. In addition to the station and district service a general field inspection routine is maintained to determine the adequacy of subordinate activities and to control the assignment of personnel. The Service Command also maintains a central laboratory to which water samples are submitted from all installations for examination. This laboratory is equivalent to a state

health department laboratory, and its reports are considered official and final. The Service Command is responsible for furnishing conculting service to field establishments, either from available Army personnel, or by contract with civilian consulting engineers, depending on the nature of the problem involved.

It is at the service command level that official liaison is established with civilian health agencies. The U. S. Public Health Service maintains a liaison officer at First Service Command Headquarters to deal with problems in public health involving the armed forces and civilian agencies. Whenever deficiencies are found in a public water supply which may endanger the public health, a report is made at once to the state health department concerned with a request for appropriate action.

The final, and superior, element in the Medical Department organization is the War Department, represented by the Office of the Surgeon General. At this level, the over-all policies of the Army are established and published as Army Regulations, War Department Circulars, or in similar form. The War Department also sets up the qualifications for officers and is responsible for procurement and assignment of qualified officers.

From this brief summary of the organization and administration of the Army water inspection service it should be evident that a close relationship exists between the Army and all public water supplies used by the Army, and by entering into a contract to provide water service, the water supply organization accepts an inspection routine superimposed on existing inspection schedules. The question immediately arises as to the standards of water purity required by the military organization. In reply to this question, the Army contracts for, and expects to receive, water that conforms to the U. S. Public Health Service standards for drinking water to be used on interstate carriers, which constitute the universally accepted criteria for drinking water. It is true that water which apparently meets these standards is sometimes declared non-potable by the Army, and may be subject to either emergency or routine chlorination prior to distribution to troops, but for the purpose of satisfying contract specifications, the Army is perfectly satisfied to accept water meeting the Public Health Service standards. Some special problems arise in this connection.

Probably the most controversial question that has arisen with regard to water supplies has been the adequacy of routine chlorination. During the summer of 1942, the War Department issued an order that all Army water supplies were to be chlorinated so as to produce a residual of at least 0.4 part per million in all active parts of the distribution systems. The reaction to this policy was violent. Many responsible engineers were not in accord with the policy, manufacturers of chlorine feed equipment did not have stocks on hand or the production capacity to meet the immediate demands for machines, a shortage of chlorine and hypochlorites was imminent, and public water supply officials were insulted at the apparent condemnation of their treatment policies. Nevertheless, the program was started. A few serious difficulties arose almost at once. Some water supplies were found, usually small ground water developments owned and operated by the Army, that would not hold a residual long enough to produce 0.4 ppm at the tap without excessive reaction or possible corrosion of the pipes. In other cases, water was chlorinated prior to storage, and it appeared to be very difficult to carry a high residual through the storage reservoirs. A great number of scattered installations, such as offices in downtown buildings, or warehouses, were included in the program by the wording of the order, and of course many public water supplies that had records of perfectly acceptable purity for many years required rechlorination by the Army before use by troops. As a result of such difficulties, the program was modified to permit four exceptions to the general policy, namely: where certain specific qualities of the water made high chlorination impossible; where an installation was located in the midst of a distribution system, such as independent offices; where water was chlorinated prior to long storage; and finally, where water was drawn from a public water supply, chlorinated prior to delivery, and with a satisfactory bacteriological history. These last conditions apply to most public water supplies, and there are very few cases where it is still necessary to rechlorinate such supplies.

It should be borne in mind that the condition of a satisfactory bacteriological history applies at the point of use by the Army establishment. It sometimes happens that sampling at a filter plant, or even in the most active portion of a distribution system, produces

very acceptable results, but when a military installation is located several miles away, on an otherwise unused part of a system, the results are not as good on samples taken at the Army meter. In such case the supply might have to be rechlorinated.

It is possible that the chlorination standard should be restated in the light of better understanding of the action of chlorine. For the past year, routine tests for free-available and chloramine residuals have been made, using the orthotolidine-sodium arsenite method, and more attention has been given to the effect of hydrogen ion concentration on the bactericidal power of chlorine. Some standards could be devised to give proper weight to such factors. However, it must be kept in mind that any standard test must be very simple, and must allow as little latitude for judgment as possible, because the test must be left in the hands of available personnel, who may be very inexperienced. Considering all things, there probably will be no change.

The value of high chlorination is very evident. Prior to its adoption, the percentage of samples containing coliform organisms in three or more of five ten-milliliter portions frequently ran to ten per cent each month. These samples included all drinking water specimens submitted to the service command laboratory. Following the adoption of high chlorination, and at the present time, this figure is running about one per cent. Practically all of the poor samples are from public supplies that are not rechlorinated.

The relation of the military establishment to local water works is not only contractual and inspectoral. In certain cases it has been to the advtantage of the Army to give direct aid to public water supplies to improve the quality of the delivered water. Furthermore, by arranging for financial or professional assistance, public supplies have sometimes been made available for use, and this has eliminated the need for expensive and difficult construction. In some cases, local water works officials have been stimulated to investigate and adopt improved treatment methods, with resulting benefit to both the military and civilian consumers.

The grant of direct aid has been less common than various coöperative arrangements. In one case, for example, a municipal water supply was unchlorinated, and consequently the water required chlorination upon delivery to a military installation. It was found that chlori-

nation of the entire supply could be more easily handled, with greater benefit to the personnel involved, than chlorination of the water at the points of delivery. A meeting was held with the board of water commissioners, and a representative of the state health department, at which it was agreed that the Army would loan a chlorinator if the municipality would operate and maintain it. In this case, the standard of chlorination was to be that ordinarily used in waterworks practice. This arrangement has proved satisfactory to all concerned.

Another type of coöperation deals with technical rather than financial or mechanical assistance, and may involve research and development. For example, in one instance the poor physical quality of a public water supply was due to apparent contamination of the raw water. A mobile trailer laboratory was dispatched, with a chemist and bacteriologist, to make a careful study of the stream. It was soon found that the principal difficulty lay in the discharge of an obnoxious industrial waste into the stream above the water plant. A further study was made by the Engineers to determine a practicable method to reduce the problem by treatment of the waste at the source, and reasonable recommendations were made through proper channels to correct the condition.

Throughout the war, there has been a desire on the part of the military establishment to understand the problems of public water supply organizations and to conduct all matters in a way to be of mutual benefit. In every instance, the waterworks officials have given ample evidence of a similar desire on their part. Faced with loss of trained personnel, hampered by shortages of critically needed equipment and material, harried by increased demands for service, and burdened with a multitude of war-time restrictions and regulations, the waterworks of New England have not only succeeded in maintaining their ordinary service but frequently have made improvements in plant operation that have resulted in a better product delivered to the consumer. It is said that no one appreciates a water supply until it is interrupted. That is not true. There are many in the Army who are close to waterworks operations who fully appreciate the outstanding contribution of public water supplies to the war effort. We know the hard work, the thankless hours, the frequent frustrations. We have added materially to the headaches in demanding better and bet-

ter service in the face of increasing difficulties in operation. Knowing these things, and knowing how the problems have been met, we pay sincere tribute to the energy and skill of the waterworks profession and acknowledge the very great part it has played and will continue to play in bringing the war to a victorious conclusion.

FALSE PRESUMPTIVE TESTS FROM WATER TREATED WITH CHLORINE-AMMONIA‡

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[*Received June 2, 1945.*]

In February, 1944, the senior author (1) reported some experiences with the chlorine-ammonia treatment of the water supply of Amherst, Massachusetts. That report was concerned mainly with a substantial increase in the incidence of false presumptive tests for coliform bacteria that followed some time after the installation of the chlorine-ammonia treatment. The present article is an account of studies made in the effort to determine the cause of the false presumptive tests and so follows logically after the first report.

EXPERIMENTAL

The bacterial cultures examined in this study were isolated from lactose broth fermentation tubes employed in the routine testing of samples from the town water supply. The samples came from several collecting stations, ranging from a few hundred yards to five miles from the chlorinating plant. The cultures were isolated and studied over a sampling period of about two years from the beginning of the chlorine-ammonia treatment. All cultures represented in the study were obtained from positive gas tubes that failed to confirm on Endo's or Eosin-Methylene-Blue media: that is, they were from false presumptive tests. Media and techniques employed in the testing conformed to Standard Methods of Water Analysis (8th ed., 1936).

The data that follow are taken from one series of cultures collected from one month's samples in the early autumn of 1943. The results obtained from these cultures are representative of those from all of the cultures studied, so it was decided to simplify this report by presenting results from one series alone selected from a sampling

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period that gave a relatively large number of false presumptive tests. The cultures isolated from the positive-gas tubes fell into two groups:

Group I. Gram-negative rods, resembling coliform bacteria, but not producing gas in lactose broth within 72 hours at 37° C.

Group II. Gram-positive aerobic sporulating rods of the genus *Bacillus*.

At relatively rare intervals coliform bacteria were isolated from the positive tubes, but this report is not concerned with them, so they are not included.

The experimental work was divided into four parts: (1) study of Gram-negative bacteria; (2) build-up of lactose-fermenting capacity of Gram-negative bacteria; (3) study of Gram-positive rods; and (4) study of fermentation by associations.

Gram-negative Bacteria. As stated previously, these bacteria (Group I) had the characteristics of coliform bacteria, except that they did not produce gas from lactose within 72 hours. Included were 25 cultures isolated from routine fermentation tests of the water supply, and also 42 cultures from similar tests of the tap water from one of the college buildings and of the untreated water from the intake reservoir of the water system. This made 67 cultures in all.

Several possibilities presented themselves: that these organisms might belong to (1) the typhoid-dysentery group, (2) the *Salmonella* group, (2) the *Proteus* genus, or (4) that they might be atypical coliform bacteria that had lost their capacity to produce gas from lactose because of their environment.

The first possibility could well be discarded, because there have been no cases of typhoid fever or dysentery in recent years in the region served by the water supply in question, as there would have been unquestionably if bacteria of these species had been present in the quantities represented by the Gram-negative cultures recovered from the water. There is the additional fact that it is usually difficult to isolate typhoid and dysentery organisms from water in which they unquestionably are present. Not content with the evidence, however, Kligler's iron agar (2) and the Wilson and Blair bismuth sulfite agar (2) were inoculated with the organisms of Group I. The result was the elimination of both the typhoid-dysentery and *Salmonella* groups as possibilities.

The question of the *Proteus* group was investigated by inoculating the cultures into tubes of sterile skim milk. In milk, bacteria of the genus *Proteus* produce a curd which they digest rapidly. Only thirteen of the cultures tested gave curd in milk, and only six of these digested the curd. Digestion was so slow that it was only partial at the end of five days. Another test employs nutrient gelatin which *Proteus* cultures liquefy rapidly, but only eight of the Group I cultures liquefied the gelatin into which they were inoculated, and liquefaction was very slow with all but one of the cultures, which gave total liquefaction in 48 hours. Thus, the probability of these organisms being species of *Proteus* was discarded.

Build-up of Gas-producing Capacity. The fourth possibility remained, that these cultures might be atypical coliform bacteria in which the capacity to produce gas from lactose had been attenuated or lost. Slow lactose fermenters are commonly encountered in sanitary investigations, and their sanitary significance remains controversial. In fact, their physiological behavior is frequently so variable that it is extremely difficult to place them reliably in any classification scheme. To determine whether or not the cultures of Group I might have temporarily lost their capacity to produce gas in lactose broth, they were inoculated serially into the medium. That is, bacteria from fresh agar-slant cultures of the organisms were inoculated into lactose broth fermentation tubes, incubated at 37° C, and examined at the end of each 24 hours for six days if they remained negative that long. At the end of the first 48 hours of incubation, fresh fermentation tubes were seeded from the preceding ones, and so on for five transfers unless gas production within 48 hours was obtained sooner. The idea was to "train" the cultures to reestablish active fermentative functions. Results may be seen in the accompanying table.

Thirteen of the cultures produced gas in lactose broth in the first transfer; that is, the second cultivation in lactose broth: one after six days incubation, three after five days, five after four days, and four after three days. Of these thirteen cultures, eleven were "trained" by the experiment to produce gas within 48 hours and two more within 72 hours. Only one culture of the eleven eventually gave gas in 24 hours in the third transfer. Of the cultures that gave gas within 48 hours, two did so in the second transfer, four more in

SUMMARY OF BUILD-UP OF GAS PRODUCTION IN LACTOSE BROTH BY GRAM-NEGATIVE BACTERIA

(Cultures were re-numbered for this table)

Culture No.	No. of transfers				Imvic reactions*				
	I	II	III	IV	Indol	M.R.	V.P.	Na citrate	Endo's* Agar
1	3*	3	1	1	—	—	+	+	A
2	3	2	2	2	—	—	+	+	At
3	3	2	2	2	—	+	—	—	A
4	3	3	2	2	—	—	+	+	A
5	4	3	2	2	—	—	+	+	A
6	4	3	2	2	—	—	+	+	A
7	4	3	2	2	—	—	+	+	A
8	4	3	3	2	—	+	—	+	A
9	4	3	3	3	—	—	+	+	At
10	5	3	3	2	—	—	+	+	At
11	5	4	3	2	—	—	+	+	At
12	5	5	3	2	—	—	+	+	At
13	6	5	3	3	—	—	—	+	A

Note: Figures in columns indicate number of days incubation required for gas production.
Imvic tests and Endo's agar reaction were made from agar slant cultures streaked from last transfer tube of lactose broth.
A on Endo's agar = resembles *A. aerogenes*.
At on Endo's agar = resembles neither *A. aerogenes* nor *E. coli*.

the third transfer, and four in the fourth transfer. No additional cultures gave gas within 48 hours in the fifth transfer. Two more of these cultures gave gas in 72 hours in the third transfer, but never within 48 hours.

While only 12.7 per cent of the cultures studied were built up to produce gas from lactose in 48 hours, this experiment does indicate that some of these cultures encountered in this particular water supply were atypical coliform bacteria which had been devitalized, so far as their gas-producing capacity was concerned, by long existence in a habitat where there was nothing that they could ferment. It seems probable that a longer series of transfers might have built up the fermentative capacity of more of these cultures. It may be observed, from the data in the table, that none of the thirteen cultures eventually giving gas were typical *E. coli* on the basis of the "Imvic tests" (3).

Gram-positive Bacteria. This group (Group II) was made up of

Gram-positive sporulating aerobic rods of the genus *Bacillus*, commonly called "hay bacillus". These were separated into sub-groups on the basis of their morphology, particularly the position of the spore and swelling of the rod, and by the liquefaction of gelatin and fermentation of dextrose, lactose, and sucrose.

Certain species of aerobic sporulating bacilli encountered in water will produce gas from lactose and consequently give false presumptive tests for coliform bacteria in water testing. For that reason the cultures of Group II, 53 in number, were inoculated into lactose-broth fermentation tubes and incubated at 37° C. Under these conditions none of these cultures produced gas.

Associations. It is well known that in making bacteriological tests of water false presumptive tests for coliform bacteria may be caused by combinations of bacterial species, usually two at a time, producing gas where none of the participants alone can do it. Such combinations are referred to as associations, and the associative action is termed a synergism or symbiosis. It was decided to try certain combinations of the organisms of Groups I and II. Two experiments were set up as follows:

In Experiment A, 25 of the cultures of Group I were combined with four key cultures representing the four main sub-groups of Group II. Lactose broth fermentation tubes were inoculated with these combinations, two organisms to each tube. Thus, each Gram-positive sporulating key culture was combined with each of the 25 Gram-negative cultures, in separate tubes. One hundred combinations resulted. The tubes were incubated at 37° C and observed each 24 hours through 96 hours. There were no positive gas results in 24 hours. One Gram-negative culture gave gas in 48 hours with all four spore-formers, and another with two spore-formers. At 72 hours, 16 additional combinations gave gas. There was no further gas production at 96 hours. The distribution at 72 hours was as follows:

3 Gram-negative cultures gave positive gas with 4 spore-formers
2 Gram-negative cultures gave positive gas with 3 spore-formers
1 Gram-negative culture gave positive gas with 2 spore-formers
2 Gram-negative cultures gave positive gas with 1 spore-former

In Experiment B, certain Gram-negative key cultures were combined with others of the same set. Earlier in the experimental work,

the Group I cultures that did not produce gas in lactose were tested as follows: reaction (pH) in lactose broth; reaction (pH) and positive or negative gas in dextrose, sucrose, and mannitol broths; test for gelatin liquefaction. On the basis of these tests, these cultures were separated into five sub-groups. For the association experiment, one key culture was selected from each of these sub-groups. Twenty cultures of the group, designated below as test cultures, were combined with each key culture, two cultures at a time, making 20 combinations for each key culture, and a total of 100 combinations. Lactose broth fermentation tubes inoculated with these combinations were incubated at 37° C and examined each 24 hours through 96 hours. Results follow:

24 hours, no gas production.
48 hours, gas production by 7 combinations.
72 hours, gas production by 19 additional combinations.
96 hours, gas production by 38 additional combinations.
Total combinations producing gas within 96 hours = 64.
One key culture produced gas in all combinations.
One key culture produced gas in all combinations but one.

These two key cultures accounted for 39 of the 64 positives.

The results for the test cultures follow:

6 test cultures gave positive gas with 5 key cultures.
3 test cultures gave positive gas with 4 key cultures.
1 test culture gave positive gas with 3 key cultures.
18 test cultures gave positive gas with 2 key cultures.
1 test culture gave positive gas with 1 key culture.

These experiments with the associations indicate that some of the false presumptive tests obtained in testing the particular water supply could have resulted from the associative action of bacteria present in the water. The combinaitons of Gram-negative cultures produced gas more actively than did the combinations of Gram-positive with Gram-negative cultures. This may be explained, possibly, by recalling that associative production of gas from lactose depends upon the ability of one of the associates to produce acid from lactose. The two key Gram-negative cultures that produced gas in all but one combination both fulfilled this condition. The other three key organisms all gave neutral or alkaline reactions in lactose broth. The

spore-formers employed as key cultures produced some acid from lactose, but the production was not so active, either qualitatively or quantitatively, as with the Gram-negative cultures mentioned.

Summary

Results obtained from the experiments here reported indicate that at least some of the Gram-negative bacteria encountered in false presumptive tests from the water supply in question were coliform bacteria whose capacity to produce gas in lactose broth had become dormant, probably as a result of environment lacking fermentable carbohydrate material. The fact that the gas-producing capacity could be built up in some of the cultures, together with the added fact that associations of cultures produced gas, suggests that these factors could very easily account for false presumptive tests frequently encountered in testing the water. Certainly, there is ample opportunity for associative action in a fluid culture medium such as lactose broth.

Some work was done to determine the effect of chlorine-ammonia on the growth and behavior of the cultures employed. Results were inconclusive. However, the fact remains that the incidence of false presumptive tests from the water increased substantially after the chlorine-ammonia treatment of the supply was begun. In fact, false presumptive tests were rarely encountered previous to the beginning of the treatment. This suggests strongly that the chlorine-ammonia treatment stimulates the growth of the responsible bacteria in the mains. Possibly the organic matter in the mains contributes to the stimulation.

Some study was made of the possibility that anaerobic bacteria may have contributed to the incidence of the unconfirmed positive gas tests. This was done because it is known that some anaerobic bacteria, particularly the Welch bacillus, may give positive gas tests at times. However, no anaerobic bacteria have been isolated so far from the water of the Amherst supply.

The authors believe that bacteria causing the false-presumptive tests encountered in testing the water are entirely saprophytic forms and that the positive tests should occasion no concern over the sanitary quality of the water.

Acknowledgment. Again the authors wish to acknowledge the

willing and generous coöperation of George C. Brehm, Superintendent, and John A. White, Foreman, of the Amherst Water Department during the experimental work.

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CHLORINE DIOXIDE FOR THE TREATMENT OF WATER SUPPLIES

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[*Read May 17, 1945.*]

Since the author first discussed chlorine dioxide before this Association (1), it has been used in several water supplies. The chief interest of this first paper was in the bactericidal effect of the dioxide, but it was suggested that this substance might be of value in the control of tastes and odors in water supplies.

It had been demonstrated that in several waters in which the application of chlorine had resulted in strong chlorophenol tastes, chlorine dioxide treatment did not produce such tastes. The dioxide has since been used successfully for the treatment of phenol-bearing waters, especially in the removal of the undesirable tastes following the applicationof chlorine for disinfection.

General Experience. Results obtained at Niagara Falls were reported by Synan in this Journal (2); and several other communities nearby have since reported similarly satisfactory results.

A report of one spectacular incident in Ontario was relayed to me through a member of this Association, F. Wellington Gilcreas, from Dr. Berry of the Ontario Department of Health: a fire in a coke pocket on the water shed of a small supply was not extinguished until large amounts of water had been pumped onto the burning fuel, and much of this water eventually entered the supply, with extremely great chlorophenol tastes as a result. The use of a high concentration of chlorine dioxide effected a complete cure in quick time.

Laboratory experiments have indicated that waters containing up to 2 ppm of phenol may be properly disinfected by chlorine and given secondary treatment with chlorine dioxide, without any resulting chlorophenol tastes. I have found, however, that with higher amounts of phenol, especially in waters containing woody or vegetable

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odors, unless treatment is rigorous, there may result a sweetish, somewhat aromatic taste, which has been described as unpleasant by some observers and as noticeable but not particularly disagreeable by others. However, the amounts of phenol, where good results have been shown, are undoubtedly far above anything to be expected in a water supply, and the resulting tastes have been at least a change for the better.

The successful use of chlorine dioxide against odors resulting from phenol led to attempts to try it in the control of other tastes, especially those described as vegetable, woody, or musty, which are quite common in New England as a result of the deciduous trees in our catchment areas and also because of woodworking and paper mill wastes in some of our streams.

In one Maine city, water was taken from a river which had a drainage area including many swamps and much cut-over woodland; the stream received wastes from several paper mills of various kinds and a small amount of domestic sewage. The water for the city supply was filtered in a modern plant, which gave satisfactory reduction of color, and other undesirable properties. In an attempt to minimize the swampy tastes and odors, split chlorination at three points in the plant was tried without much success; it was found that any attempt to carry a chlorine residual into the system resulted in considerable aggravation of the undesirable tastes, especially during warm weather. It was reported that breakpoint chlorination had not successfully reduced the tastes. In September 1944, the first experimental treatment with chlorine dioxide began. The initial dose was 0.75 ppm with about 150 per cent of the chlorine theoretically required for reaction. For several months, improvement in the water was shown at the plant, with the occasional troubles that are always encountered in exploratory work, but it was never possible to furnish a completely satisfactory water throughout the system. In midwinter, with bacterial action in the river reduced to a minimum, the contaminants gave, with the usual prechlorine treatment, an odor that was not affected by the amount of chlorine dioxide then being used, and the final water was extremely disagreeable. However, when chlorine doses were increased to give a residual of about 0.5 ppm, 1.25 ppm of chlorine dioxide completely removed the odors and tastes at the plant, and it was

found that this improvement spread rapidly through the city. At the same time, it was found that a residual of at least 0.4 ppm of chlorine could be carried throughout the system, with very little trace anywhere of the former offensive conditions.

In New Jersey, a small supply takes its water from the same river as a large city. I have received reports that the town has used chlorine dioxide with complete satisfaction while the city, which has used other treatments, is still producing an unpalatable water.

Experience at Chicopee. In Massachusetts, the city of Chicopee has struggled for several years with outbreaks of odor and taste in its raw water, generally of a musty and vegetable nature. The city's main source of supply is from a fairly shallow reservoir whose catchment area is heavily wooded, mainly with scrub oak. The city has a modern rapid filter plant with a capacity of about 5 mgd. Seasonal variations in the raw water, and the effects of flash runoff in the catchment area result in an unusually difficult treatment problem, but the color and turbidity have always been very low, and with application of quite small amounts of chlorine, it has been possible to produce water very well inside the coliform standard, although plate counts have occasionally been high. But it has been very difficult to control the odors and tastes, and during the past few winters the problem has become increasingly difficult. Carbon up to 100 lb. per mg has been applied without complete success; divided application of chlorine at several different points in the plant has only increased the odors, and ammoniation has produced little improvement and contributed to an increased growth of chromogenic bacteria.

In January 1945, the first steps were taken toward treatment with chlorine dioxide. As a result of bactericidal studies at Lawrence and elsewhere, it was decided to disregard any bactericidal action of the dioxide, and accordingly, the first step taken was to increase prechlorination doses to the point where a reasonable residual was being carried through the filters. In the meanwhile, the dioxide generator was set up and tested. The discharge tube of the chlorinator ordinarily used for post chlorination was cut and connected to the dioxide generator. Two large crocks were provided for dissolving sodium chlorite, and the solution was fed from one, while a new batch was being dissolved in the other. Changes in the amount applied were

made by changing the weight of chlorite in the crock. The solution was fed through a Var-I-Feeder tachometer controlled to a constant rate of pumpage and passes (with the chlorine solution) through a reaction chamber in which intimate mixture was assured by labyrinth flow through porcelain Raschig rings.

As soon as the increased chlorination began to give a steady residual of about 0.2 ppm through the filters, addition of activated carbon was stopped, and the use of chlorine dioxide was begun. At the start, the rate of feed was 2.5 lb. per mg of sodium chlorite and 2 lb. of chlorine, which is about twice the amount theoretically necessary for complete reaction. The resulting residual, as chlorine, was slightly in excess of 0.2 ppm, as determined by o-tolidin reaction in the usual length of time. The first point of application of the dioxide was between the sight well and the main pressure pump; later the solution was applied at the junction of the discharge lines from the filters, with a resulting somewhat longer contact period in the plant. This made control easier. Samples taken on the first day of operation showed an immediate reduction in odor, both hot and cold, and in taste; while the musty odor was not entirely removed, there was a distinct change in flavor that was a decided improvement. On the following day, the reduction was more marked, from a threshold of about 10 to about 1.5, and the character of the remaining odor was markedly more acceptable. In an effort to obtain complete removal of odor, the rate of application was doubled. Within a few days, the old musty taste and odor were almost completely eliminated, but complaints began to be made of a medicinal or chlorinous taste. These were followed by reports from consumers of tastes described as like the taste of kerosene. No such tastes had been evident at the filter plant. This observation makes it necessary to discuss a complication that, in a large measure, interfered with a completely satisfactory study of the possibility of complete removal of what might be called the normal musty odor of Chicopee water. Within the past two years, partly as a result of airplane accidents, but largely because of other circumstances, considerable amounts of various hydrocarbons had entered the upper reaches of the watershed. The presence of these substances was complicated by the use of aerosols or similar detergents, which resulted in the formation of permanent emulsions affect-

ing a large part of the water area. At times this had seriously affected the treatment of the water. When attempts were made in the laboratory to dissolve out these emulsions, the emulsifying agent or agents were so effective that organic solvents such as carbon tetrachloride were also taken into the emulsion. The concentration of hydrocarbons was so great at times that distillation of a liter of sample from the surface of the reservoir yielded several cubic centimeters of hydrocarbons. Prior to the application of chlorine dioxide, the filtration processes and activated carbon had prevented any great amount of odor or taste from these oily substances, and immediately before beginning the use of dioxide, trouble from the hydrocarbons had been insignificant. It seemed apparent from the nature and location of reports of kerosene-like tastes following the use of dioxide, that this substance had begun some sort of oxidizing action on material in the pipes of the distributing system and that this resulted in the release of hydrocarbons previously entrapped.

The dosage of 1 ppm meanwhile was quite completely removing the musty odors, but the combined chlorinous and kerosene tastes continued, and after about 10 days the dosage was reduced to 0.5 ppm of chlorine and chlorine dioxide combined. This amount continued to show a reduction in the musty odors but without giving complete removal. The hydrocarbon tastes were not much in evidence at the plant or in the nearby parts of the distribution system, but from more distant points such tastes continued to be reported. This quite definitely indicates a decided effect of the dioxide treatment on pipe slimes.

After about a month of dioxide treatment, heavy runoffs brought about such an increase in both musty and hydrocarbon odors in the raw water that it was decided to resume the use of activated carbon, and heavy applications have been continued to date. This, of course, has obscured somewhat the study of the effect of dioxide. However, the overall reduction in odors by the combined treatment has been on the whole quite satisfactory, although there was always some odor when the water was heated. On May 8, 1945, the threshold odor of the raw water was 13, and of the final effluent, 3. In the warmed samples there was a very faint hydrocarbon odor, but this was absent in the cold samples, and there was no suggestion of an oily taste.

The chlorine dioxide treatment has definitely proved of value in reducing odors and in transforming odors and tastes to less undesirable characters even if complete elimination has not been obtained. Laboratory experiments seem to indicate that complete success might have been reached if it had been possible to continue heavier treatments.

Although the burden of disinfecting the water has been placed on chlorine entirely, there has been some indication of bactericidal action from the ClO_2 in the supply. For the three months before beginning the experiment, the average plate count (20° C., 2 days) on nutrient agar was 13 per ml; in the three months following, the average was 3. No coliforms were found during either period.

Costs. The solution crocks, feed lines, and reaction tower, are relatively inexpensive; the Var-I-Feeder, too, does not involve a prohibitive capital expenditure. At Chicopee, the maximum cost for sodium chlorite and the additional chlorine for activation was about $3.60 per mg. In the Maine city, the maximum cost was about $4.50 per mg, and indications are that this will eventually be reduced to about $2.00.

Experimental Work on Tastes and Odors. We have made brief exploratory experiments on waters with another type of disagreeable taste: one supply has an average iron content of about 1 ppm, and another contains about 1 ppm each of iron and manganese. Both have an astringent taste. Application of about 1 ppm of chlorine dioxide effects very considerable improvement in the taste of both waters.

Experiments on a very small scale with another supply which has a very pronounced woody and musty taste, with a suggestion of chlorophenol, have shown great improvement with the use of ClO_2, in both odors and tastes.

Bactericidal Experiments. Previous reports have shown that chlorine dioxide has valuable bactericidal action, but only when the organic matter, as shown by the chlorine demand test, is very low. Reference should be made to the author's previous paper in this Journal for the comparative efficiency of chlorine and chlorine dioxide in a water with practically no chlorine demand and in another water relatively high in organic matter.

Although, at present, all plants using dioxide rely on pre-chlorination for disinfection, it seemed advisable to continue our studies

of the bactericidal action of the dioxide, both alone and when used in various ways in combination with chlorine.

In one series of experiments, chlorine dioxide prepared from sodium chlorite by the addition of chlorine was introduced into a water with practically no chlorine demand, to which had been added approximately 100 coliforms per 100 ml. Care was taken to prevent the formation of any chlorine residual at any time. ClO_2 evolved by the action of acid on chlorite was applied to the same water. The amounts applied were determined by titration with thiosulphate. Bacterial examinations were made periodically on all the treated samples. Table 1 shows that more than 0.5 ppm of dioxide is required to kill 100 coliforms per 100 ml, even in a water quite low in organic matter.

TABLE 1.—BACTERICIDAL ACTION OF CHLORINE DIOXIDE IN TAP WATER WITH A CHLORINE DEMAND OF ABOUT 0.10 PPM.

	Coliforms in 100 ml. for Stated ppm of ClO_2 Applied						
Time—hours	0	0.2	0.45	1.2	0.2	0.45	1.2
		(chlorine activated)			(acid-activated)		
0.5	100	10	10	0	10	6	0
1.0	100	10	10	0	10	2	0
2.0	100	10	10	0	10	2	0
4.0	100	10	10	0	10	2	0
24.0	100	10	10	0	10	0	0
		Maximum Residuals (as Chlorine), ppm					
0.17	—	.15	.19	.55	.15	.21	.70
0.5	—	.14	.19	.48	.12	.17	.65
1.0	—	.14	.20	.60	.16	.20	.60
2.0	—	.14	.17	.48	.16	.19	.55
4.0	—	.13	.18	.52	.16	.19	.55
24.0	—	tr.	.13	.45	.06	.12	.48

In another series of experiments on river water which had a chlorine demand of about 0.8 ppm, dioxide activated by chlorine was applied in sufficient amounts to give residuals of approximately 0.1, 0.2, 0.3, and 0.5 ppm with the results shown in Table 2.

To the same river water, chlorine dioxide activated by hydrochloric acid was added in amounts intended to give a somewhat wider range of residuals (see Table 3).

TABLE 2.—BACTERICIDAL ACTION OF CHLORINE DIOXIDE ACTIVATED BY CHLORINE IN RIVER WATER WITH A CHLORINE DEMAND OF 0.82 PPM.

Time—hours	Coliforms in 100 ml. for Stated ppm of ClO_2 Applied 0	1.0	1.7	2.7	3.4
0.5	10,000	100	100	8	6
1.0	10,000	100	8	6	4
2.0	10,000	10	8	6	2
4.0	1,000	6	4	2	0
24.0	1,000	2	0	0	0
	Maximum Residuals (as Chlorine), ppm				
0.17	—	.12	.18	.29	.51
.05	—	.10	.14	.20	.47
1.0	—	.05	.14	.17	.28
2.0	—	.00	tr.	.09	.13
4.0	—	.00	.00	tr.	tr.
24.0	—	.00	.00	.00	.00

TABLE 3.—BACTERICIDAL ACTION OF CHLORINE DIOXIDE ACTIVATED BY HYDROCHLORIC ACID IN RIVER WATER WITH A CHLORINE DEMAND OF 0.82 PPM.

Time—hours	Coliforms in 100 ml. for Stated ppm of ClO_2 Applied 0	1.0	2.0	3.0	6.0
0.5	1,000	100	10	6	2
1.0	1,000	100	10	10	4
2.0	1,000	100	6	6	0
4.0	1,000	100	6	4	0
24.0	1,000	10	6	0	0
	Maximum Residuals (as Chlorine), ppm				
0.17	—	.14	.40	.50	1.2
0.5	—	.13	.27	.50	1.4
1.0	—	.13	.37	.60	1.6
2.0	—	.11	.25	.50	1.4
4.0	—	.05	.16	.30	1.1
24.0	—	.00	.00	tr.	.33

These results indicate that in a water which has a chlorine demand of about 1 ppm, applications of chlorine dioxide, activated either by chlorine or by hydrochloric acid, fail to give complete kill until massive doses are used, even though considerable residuals may appear.

In a further experiment, river water with a chlorine demand of slightly less than 1 ppm was treated with sufficient chlorine to give a residual of 0.2 ppm; previous work had shown that this amount effected complete destruction of coliforms in 30 minutes. Sodium chlorite solution was added immediately after the application of chlorine in the following concentrations: 0.25, 0.50, and 0.75 ppm. Table 4 shows the coliforms and the chlorine residuals remaining after various intervals of time.

TABLE 4.—BACTERICIDAL ACTION OF CHLORINE DIOXIDE IN CHLORINATED (0.2 PPM RESIDUAL) RIVER WATER WITH A CHLORINE DEMAND OF 0.95 PPM.

	Coliforms in 100 ml. for Stated ppm of Cl_2 and ClO_2 Applied				
	0	1.15	1.15	1.15	1.15 Cl_2
Time—hours	0	0.00	0.25	0.50	0.75 ClO_2
0.5	10,000	0	4	2	2
1.0	10,000	0	2	2	0
2.0	1,000	0	2	0	0
4.0	1,000	0	0	0	0
24.0	1,000	0	0	0	0
	Maximum Residuals (as Chlorine), ppm				
0.17	—	.18	.19	.42	.72
0.5	—	.13	.18	.40	.55
1.0	—	.11	.16	.31	.55
2.0	—	.05	.11	.20	.43
4.0	—	tr.	.05	.14	.23
24.0	—	.00	tr.	tr.	.10

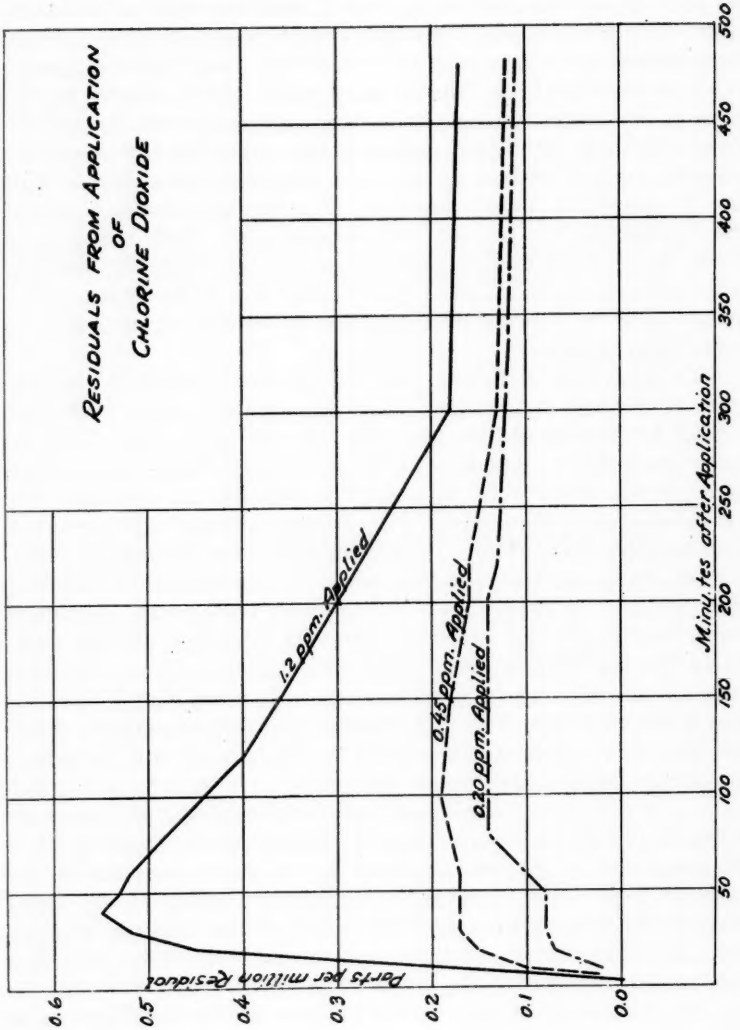
It should be noted from Table 4 that the chlorine alone effected a complete kill in 30 minutes, while chlorine dioxide slowed down disinfection, although all the residuals shown in the presence of dioxide were higher than with chlorine alone.

The bacterial experiments in general show that chlorine dioxide has definite bactericidal value, but that much larger amounts are necessary than with chlorine; also that it is important to employ pre-chlorination when the dioxide is to be used for control of odors and tastes, and sufficiently far enough ahead of the dioxide to have completed its effects. This has been recognized in all the applications thus far installed, but it is worthwhile to repeat the statement, especially as the o-tolidin residuals shown by the dioxide might give

the false impression that satisfactory disinfection could be expected. However, there are indications that small amounts of ClO_2 do have some killing effect over long periods of time, and that this should prove to be valuable in distribution systems. When chlorine is also present, the dioxide through its greater oxidizing power should continue to remove the organic materials and permit the chlorine to act upon the bacteria without expending its energy on other organic matter. There is also a fair possibility that the bactericidal action of chlorine dioxide may increase with increasing pH. Sufficient evidence is not yet at hand to give definite proof of this tendency, which is quite contrary to the action of chlorine, but if it is confirmed, it may be of real value, especially since a high pH is now maintained in so many water supplies.

Ortho-Tolidin Residuals. It was previously reported that the ordinary o-tolidin method gave sufficiently accurate results for routine control of chlorine dioxide and that the maximum color developed much more slowly than with chlorine. Careful studies were undertaken in our latest work to attempt to establish an optimum time for the reading of the residual. In the bacterial experiments reported, portions were removed for o-tolidin residuals at each bacterial examination, and other lots were run solely for the purpose of checking the residuals. In all of these, readings were made every 10 minutes for several hours, and at hourly intervals thereafter. It was again found that the maximum color developed much more slowly than with chlorine, that the time for maximum was seldom less than 30 minutes, that it was frequently 80 to 100 minutes, and that occasionally it was as long as 200 minutes. In general, the higher the ratio of dioxide to chlorine demand, the shorter the period for development of maximum color. Perhaps an easy method for obtaining maximum color would be to read the residual at 30 to 40 minutes and again at about 90 minutes after the o-tolidin is added; in most cases one or the other of these times will give the maximum reading. It was found that the maximum reading agreed well with values obtained by titration. We have used the Hallinan tests in our work, but results thus far are rather confusing and indicate the need for further study.

In the course of this work, and also in the investigation at Chicopee, we found that the residual did not completely disappear,


Residuals from Application of Chlorine Dioxide
1.2 ppm. Applied
0.45 ppm. Applied
0.20 ppm. Applied
Parts per million Residual
Minutes after Application
0.0
0.1
0.2
0.3
0.4
0.5
0.6
50
100
150
200
250
300
350
400
450
500

but in many cases, almost regardless of the amount of chlorine dioxide applied, a stable residual of about 0.12 ppm was obtained which persisted on one occasion for 9 days.

In the accompanying diagram, there can be seen the effect of dosage on the time of maximum reading, and also the tendency to approach gradually a minimum residual which remains practically unchanged for from 1 to 9 days.

Bactericidal experiments indicated that this apparent residual had very slight killing value against coliforms, but that it had some slight action against ordinary water bacteria. Chlorine in varying concentrations was added to water showing this long-lasting residual. In most cases, we found a straight-line response similar to that shown in breakpoint chlorination, but in one or two experiments, there was evidence of a secondary reaction resulting in the liberation of chlorine dioxide, indicating that the persistent residual may represent chlorite which had not been completely reduced, but had entered into some combination in which the chloride dioxide molecule was more or less unchanged.

If the maintenance of a residual were the only criterion of a safe water, here is a fairly simple way to get it. This discussion is offered as a warning to resist any temptation to rely on such a residual.

Chemical Examinations. Determinations of ammonia, nitrates, and nitrites indicated that chlorine dioxide does not readily oxdize nitrogenous matter. But the oxygen consumed analyses did show that the dioxide oxidizes carbonaceous matter. This tends to explain our earlier findings that action of the dioxide is affected much more by the organic demand of water than is chlorine. It also suggests that the efficiency of chlorine dioxide against tastes and odors may be due to its relatively complete oxidizing action, while chlorine tends to combine, both with simple substances like phenol and with more complex materials like the lignocelluloses, to give compounds that still have undesirable odors and tastes.

In any case, it seems probable that complete success in taste and odor removal is obtained when objectionable substances are more or less completely oxidized to relatively simple end products. When treatment results merely in a change of odor or taste—examples of which are the sweetish odors encountered at Chicopee or in our labo-

ratory work, or the disagreeable chlorophenol tastes—this indicates the production of substitution or addition products as well as treatment insufficient in degree or kind.

Dioxide treatment has little effect on color, unless very great amounts are employed.

Conclusions. Treatment of water supplies with chlorine dioxide can result in satisfactory removal of tastes and odors. Each supply so treated will probably require individual study, and success will be obtained only if optimum dosages, which may be high, are used. Due to oxidation of pipe slimes, tastes and odors may be increased temporarily until equilibrium is reached. Thenceforth, the treatment should have a beneficial effect on distribution systems.

Disinfection must be separated from odor treatment and should generally be completed before the application of dioxide; otherwise very high residuals must be carried.

Waters treated with dioxide will carry a high residual of chlorine without creating taste problems.

Operating costs, on the basis of our present studies, will probably be somewhat greater than costs of superchlorination, and somewhat less than costs of activated carbon.

I wish to express my appreciation to the Chicopee Water Department, especially Mr. Driscoll the Superintendent and Mr. Lavallee, the Chemist, for assistance at the plant, and to Mr. John F. Synan, of the Mathieson Alkali Works, Inc., for information secured and for our supply of sodium chlorite.

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DEVELOPMENTS IN ELECTRONIC INSTRUMENTATION

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[*Read May 17, 1945.*]

Electronics in the Instrument Field

Let us begin on a factual plane by discussing electronic instruments already on the market. In doing this I shall compare the electronic instruments with earlier mechanical instruments of the same general type, and I shall also point out the probable result of design trends that are already apparent.

In general, there is not much electronically new in the construction of electronic measuring instruments. The variables measured are voltage, resistance, inductance, and capacitance, and the bridge and potentiometer circuits used to measure them are much the same as in earlier mechanical instruments. The rather paradoxic result is that the main changes in instrument construction caused by the use of electronic techniques have been mechanical rather than electrical.

To bring this comparison into focus, let me compare Foxboro Wheatstone bridge instruments of the galvanometer type and of the newer electronic type. Both of these are so-called "null-balance" instruments, in which an unknown electrical resistance is measured by automatically balancing it against an adjustable standard resistance in the instrument to produce a resulting zero or "null" voltage.

In the mechanical instruments, the amount and direction of any lack of balance is detected by a galvanometer, which is a rather delicate and weak unit. The position of the galvanometer pointer is periodically felt by a system of sensing fingers, which then mechanically adjusts through an intermittent clutch system the standard slide-wire resistance to restore balance. The design and operation of this mechanism is much like that of a fine machine tool. Precision machine work and skilled assembly are absolute requirements during manufacture, and skilled operation and maintenance are needed to retain the original calibration accuracy during the life of the instrument.

In the electronic instrument, the galvanometer, the sensing fin-

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gers, and the intermittent clutch system are all replaced by an electronic unit with no moving parts. For direct comparison, in Foxboro electronic instruments there are only five moving parts, where the equivalent galvanometer type instruments have twenty-five.

One result of this obviously great simplification through the use of electronics is that in these new instruments there is no greasing or lubrication of gears or bearings, no cleaning or greasing of slidewires or slidewire contacts, no changing of motor brushes, and no wear and replacement of gears or cables. In fact, we believe that no mechanical maintenance whatever will normally be needed.

But this simplification and resulting elimination of maintenance is not the main advantage of using electronic techniques in instruments. The important gains are: (1) in instrument performance—dynamic measuring action with continuous high-speed response plus immunity to vibration; and (2) entirely new methods of measuring.

In galvanometer types of potentiometers and bridges, the pen speed is from 10 to 30 seconds for travel across the full chart. In electronic types, a pen speed of 3 to 5 seconds is general, and some types of instruments will give full-scale pen travel in less than one second. In galvanometer types of potentiometer recorders, the narrowest practical range is of the order of several thousandths of a volt. In one new electronic potentiometer recorder, the range for full chart is one millionth of a volt. In galvanometer types of Wheatstone bridge recorders, the narrowest temperature range has been about 20° F. In electronic types of equivalent instruments, the narrowest range is a small fraction of this same 20° spread and the limit has by no means been reached. In elastic spring pressure recorders, the top pressure which can be measured is about 20,000 psi and the lowest pressure scale width is about 1/10 in. of water. With electronic instruments, the maximum pressure that can be measured is now well over 100,000 psi and the minimum pressure is approximately 1/1000,000 in. of water. Similar comparisons might be made for other types of measurements.

With these gains in measuring speed and increased sensitivity, the measuring accuracy of all the new electronic instruments is at least as good as that of the less sensitive galvanometer types, and, in many cases, the accuracy is higher for the electronic type.

Much of this gain in performance is directly the result of the inherent characteristics of electronics. For example, a very simple vacuum tube amplifier can take a very small voltage (of the order of a millionth of a volt) and amplify this millions of times with very little distortion, with high stability, and with a very low power level required in the input to the amplifier. During the past ten years, the mechanical design of vacuum tubes has improved so that they are now very sturdy and highly immune to vibration. Also, vacuum tube action is so fast that the commonly used tubes will amplify a-c voltages with frequencies as high as hundreds of millions of cycles per second.

Looking broadly at electronic intruments with their sensitivity, accuracy, and speed far beyond old standards, we can see a broad picture developing. There is the basic idea of working toward a universal electronic instrument to measure any and all process variables. Several companies are now making progress in this direction. Another trend is the electrical measurement of variables such as pressure, liquid level, flow, and weight. This makes possible the centrallizing of a large number of measurements in a control room with simple wiring connecting sensitive elements at the points of measurement to an indicator or a recorder in the control room. With suitable selector switches, a number of different measurements can be recorded on a single instrument, or variables at many scattered points can be quickly checked with a high-speed indicator. Still another trend is the use of electrical means of continuously adding, subtracting, or taking ratios of two or more separate measurements and recording the result of this mathematical operation.

As possibly the most definite indication that these advantages are very real, I might say that out of the seven major instrument companies, six now have on the market electronic measuring instruments. But I cannot, in honesty, leave you with the picture that pressure spring instruments are fated to disappear in favor of electronic types. Overlooking for the moment the fact that these two types have different levels of performance, we feel certain that it does now and will continue to cost more to make a good electronic instrument than it does to make a good spring type pressure or temperature recorder. The apparent difference in price between mechanical instruments and

the electronic types may well be decreased by the use of mass production methods for the electronic instruments, but it is difficult to imagine that this difference will approach the disappearing point. If this assumption is correct, the use of these two types will depend on an economic choice based on the performance requirements, the first cost of the equipment, and the cost of maintenance. The immediate market for electronic instruments, therefore, is to replace the equivalent types of galvanometer instruments, such as recording potentiometers or bridges, in which mechanical balancing systems are used.

Electronics in the Water Works Industry

Having used the instrument industry as a guinea pig to demonstrate the effect of a large injection of electronics, we can now go afield and guess about what will happen in the water works field. Probably the easiest way to start is with instruments.

Measurement and Control of pH. Measurement of pH is an example of laboratory technique that has put on overalls and gone out into the factory by applying electronics. To all of us, except the technically precise, pH is a measuring unit for expressing the alkalinity or acidity of a water solution. Several types of electrodes have been developed to measure this variable by producing an output voltage related to the pH value of the solution. Of these various electrodes, the glass electrode has been developed today to a point where it is the most widely useful. But glass electrodes have one great problem: their very high electrical resistance and very low permissible output current. Today, vacuum tube amplifier units have completely superseded, for general plant use, the sensitive and very delicate light beam galvanometers first used in the laboratory.

During the past fifteen years, reliable and sturdy glass electrode ph instruments of both portable indicating types for laboratory use and continuous recording types for plant use have been developed. With no exception that I know about, all of the designs have been based on the use of electronics. The water works industry has a large stake in the successful development of these instruments. Some of you use water into which industrial wastes varying greatly and erratically in quantity and chemical strength are discharged. These wastes can upset filter plant action or cause organic growths. Meas-

urement of pH is the most powerful tool available to control the neutralizing of chemical wastes or the chemical dosing of organic wastes in treatment plants.

Telemetering. It is probable that there will also be an increasing use of electronic techniques in telemetering equipment for the transmission of measurements of flow, level, and other variables over long distances. It has been our own opinion that all telemetering equipment up to date has been rather unsatisfactory. The reason for this has lain not in the transmitting and receiving instruments but rather in the vulnerability of the transmission line to many sorts of failure. In general, telephone circuits or special 2-wire circuits have been used for transmission of these measurements. Typical troubles are line failures caused by lightning, breakage of wires by snow, wind, or falling trees, short circuits or damage by birds or animals, and malicious damage by human beings.

The obvious function of electronics in telemetering is to eliminate the transmission line with its failures and to use radio transmission instead. This equipment should be available immediately after the war, as all the engineering and application problems have already been worked out. Radio telemetering systems have been in use in the far West for some years for the measurement of reservoir levels and dam gate positions. The transmitters are unattended, battery-powered units, which transmit readings at regular intervals. The batteries are replaced about once every six months, and the transmitter is inspected and serviced then. Two companies now have on the market equipment of this type.

An even more convincing proof of the reliability and sturdiness of unattended radio transmitters is that the Coast Guard is now using radio beacon buoys. These transmit radio code signals, which are picked up by direction finders on boats. These transmitters again are unattended except for periodic battery replacement. I think you will agree that this service is more severe than any found in a water system.

Another type of electronic telemetering, which has proved very reliable, is carrier current equipment. This is guided radio transmission. A weak radio signal is coupled to a power transmission line, along which it travels. Several such signals can be transmitted over

the same wire without interference with each other or with the flow of electric power in the same line. These radio signals will jump short gaps in the line, such as open switches, breaks, or grounds.

Flow Metering. For measuring flow rates and total flows, the use of Venturi tubes with differential-pressure manometer instruments has long been standard in water works. It is my belief that somebody is going to put on the market in the next few years a main line meter that will use neither a Venturi tube or orifice plate nor a mercury manometer instrument.

The principle of this new flow meter has already been proved, and patents have been issued. The principle involved is the Doppler effect. You all know how the apparent pitch of a locomotive whistle is different when the locomotive is coming towards you and going away from you. This change of apparent pitch is directly related to wind velocity and locomotive speed. Exactly the same principle applies to the transmission of sound in water.

The tremendous development in underwater signalling and location equipment during the last few years has very probably worked out all the engineering problems, such as the electronic generation and transmission of pulses of sound waves, the detection of these pulses, and the conversion to flow velocity from apparent frequency shift.

Turbidimeters. Turbidimeters of various types have been described in the technical literature during the past few years. There are on the market several commercial units that use photo-electric cells. But their use seems to be developing rather slowly. Possibly the answer is a lack of general data on what the measurements with these instruments mean and just how they can be used in plant control.

Titrometers. For routine chemical laboratory analyses, titrometers using electronic voltage measuring circuits have been invented and reinvented in considerable numbers over the past five years. At present, each of the larger laboratory supply houses seems to have its own unit, which is distinctive in detail but similar in principle to the others.

Flocculation. I am personally highly optimistic about the use of electronic equipment directly in accelerating or modifying separation processes and chemical reactions. The influencing of these

processes by direct application of electrical energy in the form of high frequency electric fields or spark discharges or in the form of high frequency sound waves is just now on the way from being a laboratory curiosity to becoming a laboratory success.

Of the three various possible methods just mentioned, the use of high frequency (or so-called supersonic) sound waves is promising for separation processes, such as sedimentation or flocculation. The equipment used is a vacuum tube oscillator, which generates a-c power at the desired frequency (commonly around 50,000 cycles per second). This power is fed to a converter unit, such as a piezo-electric crystal or a magnetically driven metal bar, which is immersed in a liquid. The crystal or bar vibrates at the frequency of the oscillator and thus sets up compression waves in the liquid. This action produces a number of effects in the liquid. In some cases, it can produce emulsions of two liquids usually considered mutually immiscible. Some chemical reactions are accelerated.

The particular effect of interest to water works engineers is that of flocculation. In suspensions containing particles with specific gravities only slightly more than that of the liquid in which they are suspended and with a size above a certain minimum, flocs are formed as soon as the supersonic vibrations begin. With gas in the solution, the gas is forced into bubbles of increasing size and so expelled from the liquid. The suspended particles are driven to these bubbles by the pressure of the waves and so the particles are also expelled.

To me, there is in this supersonic equipment, the possibility of a radical change in filter plant methods and practice. It could mean either the total elimination, or else a great decrease in the size of sand filter beds used, and the employment of small electronic flocculation units through which the raw water would flow at relatively high velocities.

Disinfection. Still another field in which the use of electronic equipment may become of major importance is that of disinfecting water. Chlorination has been one of the accepted means of the past. One of the most recent advances is "break point" chlorination, which depends on continuous measurement of the pH of the treated water and regulation of the rate of dosing with chlorine gas by means of this measurement. As mentioned before, in discussing pH measure-

ment, electronics becomes an essential part of this equipment, therefore.

A far more radical possibility is the use of special lamps to sterilize water by passing through it light rays in the ultra-violet zone, which kill many microörganisms. This technique is already being tried on a commercial scale in hospitals, medical supply plants, and food processing.

Still another possibility is the use of supersonic sound waves of the type previously described. In addition to the physical and chemical effects mentioned before, these waves also have a powerful disintegrating effect on microörganisms in the liquid in which the waves are generated. This particular effect has probably had more study than any other possible application of supersonic energy, and as it is reported that some of the under-water detection equipment now in use is based on the use of supersonic frequencies, almost certainly there are new and powerful units ready for further developments in this field.

Pipe Location and Testing. A field of rather specialized interest to water supply operating men is that of underground pipe locating. Before the war, several pipe locators based on well known electronic principles were in use. The wide use of high-frequency field electronic units to locate buried mines during the war makes us certain that further improvements in this equipment will be available soon after the war.

ANALYSIS OF FLOW IN PIPE NETWORKS BY METHOD OF BALANCING FLOWS

BY CLARENCE E. CARTER* AND SCOTT KEITH*

[*Received April 19, 1945.*]

Since the introduction by Professor Hardy Cross in 1936 (1) of the method of successive approximations for analyzing the flow in pipe networks, both by the method of Balancing Heads and the method of Balancing Flows, there have been several subsequent simplifications in the mechanics of applying this principle to but one of these methods—that of Balancing Heads. Such, for example, was the method of tabulating the computations proposed by Gordon M. Fair (2 and 3). There have been, however, no published papers relating to simplifications for the method of Balancing Flows.

The method of Balancing Heads is, according to Cross (1), "directly applicable where the quantities flowing at inlets and outlets are known". There are, however, cases where the problem of analysis involves several points of inflow, and it is desired that the proportion of the total flow contributed at each of such points be determined. Under such conditions, the method of Balancing Heads is not applicable, as it will be found that regardless of what proportion of inflow is assumed at each point, this method will result in routing the flow throughout the network with no change whatsoever in the assumed proportions.

For such a problem, where the *heads* at inlets and outlets are known, the method of Balancing Flows is applicable. This was pointed out by Cross (1).

The suggested tabulation for enumerating the given data and computing the results of each successive correction to the assumed values is in no way a modification of the principles conceived by Hardy Cross, but is only a convenient means of arriving at the result desired which is believed to be better adapted to routine office computation than the method of bookkeeping proposed by Professor Cross.

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Figure 1 illustrates the method of tabulation and computation. The following procedure is suggested:

a. Make a sketch of the system under consideration. Assign a number or a letter to each junction of the network, including the points of inflow and outflow.

b. Tabulate the given data as follows:

In Column 1 list each junction point, omitting junctions where inflow and outflow occur, except in cases of inflow from a pipe within the grid as well as from a main feeder (as at "B" in Figure 1).

In Column 2 list the pipes involved at each of the tabulated junctions (as at Junction B the various pipes BA, BE and BC).

In Column 3 show the pipe length, in feet.

In Column 4 show the pipe diameter, in inches.

In Column 5 show the Hazen-Williams coefficient C, either assumed values or actual values found by test.

In Column 6 show r_o, the loss of head in feet per thousand feet of pipe for a unit flow of say 1.0 mgd. These losses may be obtained from "Hydraulic Tables" by Williams and Hazen, or they may be computed by use of the Hazen-Williams slide rule. Values of r_o for several values of the Hazen-Williams C and for a range of pipe sizes varying from 4 to 30 in. in diameter are given in appended Table A for convenience.

In Column 7 calculate the resistance of pipe, in feet, $r = r_o \times$ length, or Column 3 $\times$ Column 6.

c. Assume values of the head at each junction, based on the given heads at inlets and outlets:

In Column 8 list the head (assumed), in feet. The tabulated value represents the drop in hydraulic gradient for the particular pipe under consideration between the two junctions.

In Column 9 calculate h/r, or Column 8 $\div$ Column 7.

In Column 10, list Q, in mgd, where $Q = \left(\frac{h}{r}\right)^{0.54}$. These values must be tabulated with due regard to signs. Flow toward the junction is given a + sign; flow away from the junction a — sign. Values of numbers raised to the 0.54 power are given for convenience of calculation in appended Table B.

In Column 11 show $Q^{0.85}$. Values of numbers raised to 0.85

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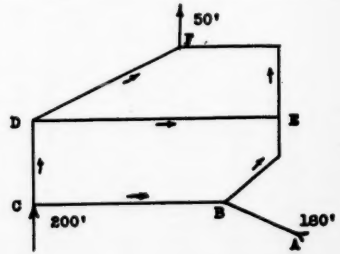
ANALYSIS OF FLOW IN PIPE NETWORK 3.
(METHOD OF BALANCING FLOWS)

PLACE **Sherman, Wis.**
DISTRICT **Cherry Valley**
COMPUTED BY **C.E.C.** DATE **1/45**



Flows towards junction are +
Flows away from junction are −
(a) r_o is loss of head per 1,000 ft for unit flow (1.0 m.g.d.)
(b) $R = \frac{1}{1.85\, r Q^{0.85}} = \frac{0.54}{\text{Col. 7} \times \text{Col. 11}}$ $\quad \Delta = -\left(\frac{R}{\Sigma R} \times \Sigma Q\right) = -$ Col. 13 × Σ Col. 10 (with regard to signs)

Corrections in Δ′ column are those computed for same pipe in companion circuit, with opposite signs.

Junction	Pipe	Length in Ft.	Diam. in In.	H-W C	r_o (1 m.g.d.)	r ($r_o \times L$)	h (Assumed)	$\frac{h}{r}$	$Q = \left(\frac{h}{r}\right)^{0.54}$	$Q^{0.85}$	$R = \frac{1}{1.85 r Q^{0.85}}$	$\frac{R}{\Sigma R_{/o}}$	Δ	Δ′	Q_1	Δ	Δ′	Q_2	Corrected $\frac{R}{\Sigma R_{/o}}$	Δ	Δ	Q_3	Δ	Δ′	Q_4	Δ	Δ′	Q_5
	2	3	4	5	6	7	8	9	10	11	12	13	14		15	16		17	18									
B	BA	500	12	120	1.50	0.75	30	40	+7.33	5.45	.138	71	−2.75		+4.58	+0.61		+3.97	80	+0.54		+3.63	+0.10		+3.53	+0.04		+3.49
(a)	BE	1000	10	120	3.64	3.64	50	13.8	+4.12	3.33	.045	25	−0.97	+0.81	+4.22	+0.22	−0.45	+4.01	17	+0.07	−0.12	+3.96	−0.02	+0.05	+3.93	+0.01	−0.01	+3.93
	BC	1800	6	100	61.0	109.8	50	0.46	+0.66	0.70	.007	4	−0.15		+0.51	+0.04		+0.47	3	+0.01		+0.46	−0.01		+0.45	0		+0.45
									+5.87		.184				+0.87			+0.43				+0.15			+0.05			
D	DC	600	10	120	3.64	2.18	40	18.3	+4.80	3.79	.065	68	+0.26		+5.06	−0.15		+4.91	65	+0.07		+4.84	−0.03		+4.81	+0.02		+4.79
(b)	DE	2200	8	100	15.1	33.2	60	1.8	+1.37	1.31	.012	12	+0.05	+0.21	+1.10	−0.03	−0.11	+1.02	16	+0.02	−0.04	+1.00	−0.01	+0.02	+0.98	0	0	+0.98
	DF	1800	10	100	5.10	9.18	110	12.0	+3.82	3.12	.019	20	+0.08		+3.74	−0.04		+3.78	19	+0.02		−3.80	−0.01		+3.81	0		+3.81
									+0.39		.096				+0.22			+0.11				+0.04			+0.02			
(a)	EB	1000	10	120	3.64	3.64	50	13.8	+4.12	3.33	.045	47	+0.87	+0.91	+4.22	−0.43	−0.22	+4.01	49	+0.12	+0.07	+3.96	−0.05	+0.02	+3.93	+0.01	−0.01	+3.93
(b)	ED	2200	8	100	15.1	33.2	60	1.8	+1.37	1.31	.012	12	+0.22	−0.08	+1.10	−0.11	−0.05	+1.02	17	+0.04	+0.02	+1.00	−0.02	+0.01	+0.98	0	0	+0.98
	EF	900	10	100	5.10	4.59	50	10.9	+3.64	3.00	.039	41	+0.76		+4.40	−0.38		+4.78	34	+0.08		+4.86	−0.03		+4.89	+0.01		+4.90
									+1.85		.096				+0.92			+0.25				+0.10			+0.02			

FIGURE 1

power are given for convenience in appended Table C.

In Column 12 calculate $R = \frac{1}{1.85\, r\, Q^{0.85}} = \frac{0.54}{\text{Col. 7} \times \text{Col. 11}}$

In Column 13 determine $\frac{R}{\Sigma R}$ This column is used for subsequent corrections until R is corrected again in Column 18. The value of R should be recomputed after each adjustment of flows, but it has been found that with one correction after two adjustments the computations are simplified without material difference in the resulting distributions of flows.

In Column 14, find

$$\Delta = -\frac{R}{\Sigma R} \times \Sigma Q = \text{Col. 13} \times (-\Sigma \text{ Col. 10})$$

It should be noted that the minus sign in the equation for Δ (Col. 14) represents a counterflow. If, for example, at any junction the value of ΣQ (the sum of the quantities tabulated in Col. 10), with due regard to signs, shows an excess of plus quantities, the values of the corrections Δ are used to reduce the various values of Q, and are minus quantities.

In pipes common to two junctions it will be found convenient to mark each pair of pipes with a number or designating symbol in the margin to the left of Column 1. In such pipes common to two junctions two corrections must be applied, one for each junction. The correction Δ' is the correction from the other junction with the sign changed.

In Column 15 determine $Q_1 = Q + \Delta + \Delta'$.

At any junction the inflow must equal the outflow, or $\Sigma Q = 0$. The results of the analysis given in Figure 1 may be interpreted as follows:

Flow at Junction B

Inflow, in pipe AB	+3.49 mgd
Outflow, Pipe BC	+0.45 mgd
Pipe BE	—3.93
	—3.48 mgd

Flow at Junction C The total inflow is equal to the total of flows away from C, or:

Flow in pipe CB	—0.45 mgd
Flow in pipe CD	—4.79
	—5.24
Flow to C from outside the network is	5.24 mgd

Flow at Junction F The outflow from Junction F. is equal to the total of the flows coming into Junction F:

Flow in pipe DF	—3.81 mgd
Flow in pipe EF	—4.90
Total	—8.71 mgd

The inflow to the network is equal to the inflow at Junction A plus the inflow at Junction C, or $3.49 + 5.24 = 8.73$ mgd, which is in reasonable agreement with the outflow at Junction F of 8.71 mgd.

It may be advantageous, in some cases, to express the inflows and outflows in percentages of the total flow, thus:

Junction B (Inflow)	3.48 mgd or	40% of total flow
Junction C (Inflow)	5.24 mgd or	60% of total flow
Junction F (Outflow)	8.72 mgd or	100% of total flow

Table 1 has been prepared to illustrate the convergence in the successive corrections in this problem.

TABLE 1.—SUMMARY OF FLOWS IN FIGURE 1

Pipe	Flow in mgd						Per cent
	Original	1st	2nd	3rd	4th	5th	of total
Inflow at A							
BA	7.33	4.58	3.97	3.63	3.53	3.49	40.1
Inflow at C							
CD	4.80	5.06	4.91	4.84	4.81	4.79	54.8
CB	0.66	0.51	0.47	0.46	0.45	0.45	5.1
Total	5.46	5.57	5.38	5.30	5.26	5.24	59.9
Total Inflow	12.79	10.15	9.35	8.93	8.79	8.73	100.0
Outflow at F							
DF	3.82	3.74	3.78	3.80	3.81	3.81	43.6
EF	3.64	4.40	4.78	4.86	4.89	4.90	56.4
Total	7.46	8.14	8.56	8.66	8.70	8.71	100.0

The head lost in any pipe in the network may be computed from the balanced discharges (tabulated in Figure 1 as Q_5) either by the Hazen-Williams slide rule from the data in Columns 1, 2 and 3 or from the relation $H = rQ^{1.85}$, using the values of r tabulated in Column 7. The total losses from inlets to outlets along the various possible routes may then be compared to ascertain the degree of accuracy resulting from the successive corrections in the flows. Table 2 gives the results of such computation:

TABLE 2.—COMPARISON OF COMPUTED AND ASSUMED HEAD LOSSES

Pipe	Discharge Q_5 mgd	Corresponding Loss of Head, Feet	Assumed Loss of Head Feet
BA	3.49	7.5	30
BE	3.93	46	50
BC	0.45	25	50
DC	4.79	40	40
DE	0.98	32	60
DF	3.81	110	110
EB	3.93	46	50
ED	0.98	32	60
EF	4.90	87	50

From Table 2, it is seen that the head at Junction B is 175 ft., as computed from Junctions C to B, and 172 ft. as computed from Junctions A to B. The head at Junction E is 128 ft. as computed from Junctions C to D to E and 126 ft. as computed from Junctions A to B to E. Similarly the head at the outlet, Junction F, is 50 ft. as computed from Junctions C to D to F, and 40 ft. as computed from Junctions B to E to F. There is, therefore, a difference of 10 ft. in head at Junction F as computed from the given heads at the two inlets through the two routes possible to this point. Further corrections might be made and this error reduced. The percentage of inflow at the various inlets will be found, however, to vary but little from that found after five corrections. In cases where the convergence is not satisfactory it is suggested that the percentages of inflow and outflow at the given inlet and outlet points be determined and with these

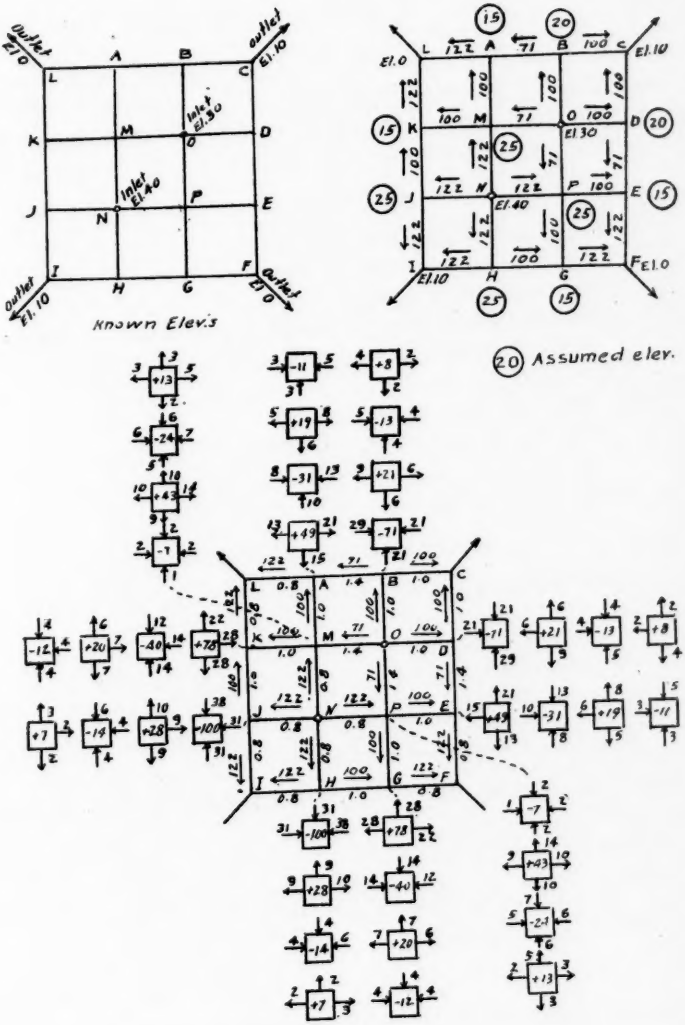

Outlet El.10
A
B
Outlet El.10
L
C
Inlet El.30
K
M
O
D
Inlet El.40
J
N
P
E
I
H
G
F
Outlet El.10
Outlet El.10
Known Elev.s
El.0
El.10
El.30
El.40
El.10
El.0
Assumed elev.

FIGURE 2

data the final adjustment throughout the network be made by the method of balancing heads.

Figure 2 shows Problem No. 7 of the original publication by Professor Hardy Cross (1). Steps are carried forward in round numbers, and no detail of procedure is shown. The following assumptions were made in this problem:

$h = rQ^2$

$r = 0.001$

Resistances in all pipes are the same.

Lengths of all pipes are the same.

Elevations are as noted.

Flows are distributed to the pipes connecting at the junctions in inverse ratio to their resistances, hence:

$R = (nrQ^{n-1})$

From these assumptions:

Flows are given by: $Q^2 = h/r = h \times 1000$, or:

h	Q
15	$(15 \times 1000)^{0.5} = 122$
10	$(10 \times 1000)^{0.5} = 100$
55	$(5 \times 1000)^{0.5} = 71$

Distribution ratios for unbalanced flows are as follows:

For $Q = 122$,	$100/122 = 0.8$
For $Q = 100$,	$100/100 = 1.0$
For $Q = 71$,	$100/71 = 1.4$

Flows are distributed as $1/R$ or $1/(nQ^{n-1})$, or since $n = 2$ and $r = 0.001$ are constant in all cases, r varies as $1/Q$ or as $100/Q$; hence distributing ratios are as given.

The final adjusted distribution of flows obtained after four corrections is:

Inlet N	537	59%	Outlet L	268	30%
Inlet O	361	41%	Outlet C	166	18%
			Outlet F	268	30%
Total	898	100%	Outlet I	196	22%
			Total	898	100%

Regarding this particular problem Professor Cross states: "The convergence here is slow and not very satisfactory, partly because the

first guess as to intermediate heads is not very good. Nevertheless, the computations indicate quickly the approximate distribution of flow at inlets and outlets."

When this problem is tabulated in accordance with the procedure illustrated in Figure 1, the final distribution of flows is as follows:

Inlet N	526	59%	Outlet L	262	30%
Inlet O	362	41%	Outlet C	168	19%
			Outlet F	262	29%
Total	888	100%	Outlet I	196	22%
			Total	888	100%

The necessary calculations are shown in Figure 3.

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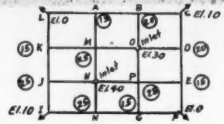


Junction	Pipe	Length ft	Diam in.	H&W C	r_c (1 mgd)	$r_c \times L$	r	h Assumed	h/r	Q (+)	R/d	R ΣR %	Δ	Δ'	Q_1	Δ	Δ	Q_2	R ΣR %	Δ	Δ'	Q_3	Δ	Δ	Q_4	Adjusted Q
A	AL						0.001	15	15000	-122	0.8	26%	-12		-134	+8		-126	19%	-3.5		-129.5	+2		-127.5	-128
(a)	AM						"	10	10000	+100	1.0	31	-15	-2	+83	+10	+10	+103	23	-4.5	-5	+93.5	+3	+2	+98.5	
(b)	AB						"	5	5000	+ 71	1.4	44	-22	-29	+20	+13	+9	+42	58	-11	-7	+24	+7	+6	+37	
										+49					-31			+19				-12			+8	
B(a)	BA						0.001	5	5000	- 71	1.4	41	+29	+22	- 20	-9	-13	-42	54	+7	+11	- 24	-6	-7	- 37	
	BO						"	10	10000	+100	1.0	29.5	+21		+121	-6.5		+114.5	20	+2.5		+117	-2		+115	+116
	BC						"	10	10000	-100	1.0	29.5	+21		-79	-6.5		-86.5	26	+3.5		-82	-3		-85	-84
										-71					+22			-13				+11			-7	
D	DC						0.001	10	10000	-100	1.0	29.5	+21		-79	-6.5		-85.5	26	+3.5		- 82	-3		- 85	-84
	DO						"	10	10000	+100	1.0	29.5	+21		+121	-6.5		+114.5	20	+2.5		+117	-2		+115	+116
(c)	DE						"	5	5000	- 71	1.4	41	+29	+22	- 20	-9	-13	-42	54	+7	+11	- 24	-6	-7	- 37	
										-71					+22			-13				+11			-7	
E(c)	ED						0.001	5	5000	+ 71	1.4	44	-22	-29	+20	+13	+9	+42	58	-11	-7	+24	+7	+6	+37	
(d)	EP						"	10	10000	+100	1.0	31	-15	-2	+83	+10	+10	+103	23	-4.5	-5	+93.5	+3	+2	+98.5	
	EF						"	15	15000	-122	0.8	25	-12		-134	+8		-126	19	-3.5		-129.5	+2		-127.5	-128
										+49					-31			+19				-12			+8	
G	GF						0.001	15	15000	-122	0.8	28	-22		-144	+11		-133	21	-4		-137	+3		-134	-134
(e)	GP						"	10	10000	+100	1.0	36	-28	-2	+ 70	+14.5	+10	+94.5	30	-6	-5.5	+83	+4	+2.5	+87.5	
(f)	GH						"	10	10000	+100	1.0	36	-28	-38	+ 34	+14.5	+10	+58.5	49	-10	-7.5	+41	+6	+5	+52	
										+78					-40			+28				-13			+7.5	
H(f)	HG						0.001	10	10000	-100	1.0	38	+38	+28	- 34	-10	-14.5	-58.5	51	+7.5	+10	-41	-5	-6	-52	
	HN						"	15	15000	+122	0.8	31	+31		+153	-9		+144	20	+3		+147	-2		+145	+145
	HI						"	15	15000	-122	0.8	31	+31		- 91	-9		-100	29	+4		-96	-3		-99	-98
										-100					+28			-14.5				+10			-6	
J	JI						0.001	15	15000	-122	0.8	31	+31		- 91	-9		-100	29	+4		-96	-3		-99	-98
	JN						"	15	15000	+122	0.8	31	+31		+153	-9		+144	20	+3		+147	-2		+145	+145
(g)	JK						"	10	10000	-100	1.0	38	+38	+28	- 34	-10	-14.5	-58.5	51	+7.5	+10	- 41	-5	-6	- 52	
										-166					+28			-14.5				+10			-6	
(g)	KJ						0.001	10	10000	+100	1.0	36	-28	-38	+34	+14.5	+10	+58.5	49	-10	-7.5	+41	+6	+5	+52	
(h)	KM						"	10	10000	+100	1.0	36	-28	-2	+70	+14.5	+10	+94.5	30	-6	-5.5	+83	+4	+2.5	+87.5	
	KL						"	15	15000	-122	0.8	28	-22		-144	+11		-133	21	-4		-137	+3		-134	-134
										+78					-40			+28				-13			+7.5	
M(h)	MK						0.001	10	10000	-100	1.0	24	+2	+28	-70	-10	-14.5	-94.5	23	+5.5	+6	-83	-2.5	-4	-89.5	
(a)	MA						"	10	10000	-100	1.0	24	+2	+15	-83	-10	-10	-103	21	+5	+4.5	-93.5	-2	-3	-98.5	
	MO						"	5	5000	+ 71	1.4	33	+2		+73	-14		+59	37	+9		+68	-4		+64	+65
	MN						"	15	15000	+122	0.8	19	+1		+123	-9		+114	19	+5		+119	-2		+117	+118
										-7					+43			-24.5				+10.5			-7	
P	PO						0.001	5	5000	+ 71	1.4	33	+2		+73	-14		+59	37	+9		+68	-4		+64	+65
(d)	PE						"	10	10000	-100	1.0	24	+2	+15	-83	-10	-10	-103	21	+5	+4.5	-93.5	-2	-3	-98.5	
(e)	PG						"	10	10000	-100	1.0	24	+2	+28	-70	-10	-14.5	-94.5	23	+5.5	+6	-83	-2.5	-4	-89.5	
	PN						"	15	15000	+122	0.8	19	+1		+123	-9		+114	19	+5		+119	-2		+117	+118
										-7					+43			-24.5				+10.5			-7	

Resistances same in all pipe runs

Figure 4

APPENDIX

TABLE A.—VALUES OF r_o FOR USE IN HYDRAULIC NETWORKS ANALYSIS BASED ON HAZEN-WILLIAMS' FORMULA

Diameter of Pipe Inches	r_o = Loss of Head in Feet per 1,000 Feet for a Discharge of 1 mgd				
	C=80	C=90	C=100	C=110	C=120
4	668	537	442	370	315
6	92.1	74.1	61.0	51.1	43.5
8	22.8	18.3	15.1	12.7	10.8
10	7.70	6.20	5.10	4.27	3.64
12	3.17	2.55	2.10	1.76	1.50
14	1.51	1.22	1.00	0.838	0.714
16	0.79	0.632	0.52	0.436	0.371
18	0.440	0.354	0.291	0.244	0.208
20	0.263	0.211	0.174	0.146	0.124
24	0.109	0.087	0.072	0.060	0.051
30	0.036	0.029	0.024	0.020	0.017

TABLE B.—VALUES OF THE 0.54 POWER OF NUMBERS

N	0	1	2	3	4	5	6	7	8	9
0	0.0	1.0	1.5	1.8	2.1	2.4	2.6	2.9	3.1	3.3
10	3.5	3.7	3.8	4.0	4.2	4.3	4.5	4.7	4.8	4.9
20	5.0	5.2	5.3	5.4	5.5	5.7	5.8	5.9	6.0	6.1
30	6.3	6.4	6.5	6.6	6.7	6.8	6.9	7.0	7.1	7.2
40	7.3	7.4	7.5	7.6	7.7	7.8	7.9	8.0	8.1	8.2
50	8.3	8.4	8.5	8.5	8.6	8.7	8.8	8.9	8.9	9.0
60	9.1	9.2	9.3	9.3	9.4	9.5	9.6	9.7	9.7	9.8
70	9.9	10.0	10.1	10.1	10.2	10.3	10.4	10.4	10.5	10.5
80	10.6	10.7	10.8	10.8	10.9	11.0	11.0	11.1	11.2	11.2
90	11.3	11.4	11.4	11.5	11.6	11.7	11.7	11.8	11.9	11.9
100	12.0	12.0	12.1	12.2	12.3	12.3	12.4	12.5	12.6	12.7
110	12.7	12.7	12.8	12.9	12.9	13.0	13.1	13.1	13.2	13.3
120	13.3	13.4	13.4	13.5	13.5	13.6	13.7	13.7	13.7	13.8
130	13.8	13.9	13.9	14.0	14.0	14.1	14.2	14.2	14.3	14.3
140	14.4	14.5	14.5	14.6	14.6	14.7	14.8	14.8	14.9	14.9
150	15.0	15.1	15.1	15.2	15.2	15.3	15.3	15.4	15.4	15.5

TABLE C.—VALUES OF THE 0.85 POWER OF NUMBERS

N	0	1	2	3	4	5	6	7	8	9
0	0.0	1.0	1.8	2.5	3.2	3.9	4.6	5.2	5.9	6.5
10	7.1	7.7	8.3	8.8	9.4	10.0	10.6	11.1	11.6	12.2
20	12.8	13.2	13.8	14.4	14.9	15.4	15.9	16.4	16.9	17.4
30	18.0	18.5	19.0	19.5	20.0	20.5	21.0	21.5	22.0	22.5
40	23.0	23.5	24.0	24.4	24.9	25.4	25.9	26.4	26.8	27.3
50	27.8	28.3	28.7	29.2	29.7	30.1	30.6	31.1	31.6	32.0
60	32.4	32.9	33.3	33.8	34.2	34.7	35.1	35.6	36.0	36.5
70	37.0	37.4	37.9	38.3	38.7	39.1	39.6	40.0	40.5	41.0
80	41.5	42.0	42.4	42.8	43.3	43.7	44.1	44.5	45.0	45.4
90	45.8	46.3	46.7	47.1	47.6	48.0	48.4	48.8	49.2	49.6

SECOND PROGRESS REPORT OF THE COMMITTEE ON RAINFALL AND YIELD OF WATERSHEDS IN NEW ENGLAND*

September 24, 1940.

To the New England Water Works Association, Gentlemen:

At the meeting of the Executive Committee of the Association on November 17, 1938, the desirability of bringing the data on rainfall and yield of watersheds up to date was discussed and it was voted:

"That the president be empowered to appoint a Committee on Rainfall and Yield of Watersheds in New England."

President George W. Sampson appointed the following as members of this committee: Harold K. Barrows, Paul L. Bean, Richard S. Holmgren, Harvey B. Kinnison, Elbert E. Lochridge, Arthur T. Safford, Caleb M. Saville, Charles W. Sherman, Miner R. Stackpole, James L. Tighe, and Arthur D. Weston, *Chairman.*

The papers by the late X. H. Goodnough, former Chief Engineer of the Massachusetts Department of Public Health, prepared with the assistance of R. M. Whittet, H. R. Crohurst, G. V. White and other members of his staff, and published in the Journal of the New England Water Works Association of September, 1915, September, 1921, June, 1926, and June, 1930, form the foundation for preparing the portion of the present report which deals with rainfall in New England and the Committee has been fortunate in having been able to persuade George V. White to join the committee in this portion of its work.[1]

The thorough report of the Committee on Yield of Drainage Areas published in the December, 1914, Journal of the Association during the extended dry period, which began in June, 1908, forms an

*EDITOR'S NOTE: The second and third progress reports of this Committee of the Association are printed in this issue of the Journal because preparation of a final report has been delayed by the exigencies of the war and because the data so far accumulated by the Committee are believed to be of current interest to the members of the Association.

[1]EDITOR'S NOTE: Mr. White has since published the available rainfall records for New England in this Journal as follows: December 1942, p. 405, March 1943, p. 15, and September 1943, p. 201.

excellent starting place from which to proceed in the preparation of the portion of the present report dealing with the Yield of Watersheds. Francis H. Kingsbury who has given much time to the preparation of material on the yield of drainage areas has also been added to our committee.

The Committee is informed that it is the desire of the Executive Committee to bring into one report the basic information dealing with rainfall together with a report on the yield of watersheds resulting therefrom.

In any locality, the yield of watersheds depends upon the precipitation and varies with the amount, seasonal distribution, and character of precipitation, subject to such losses as may be experienced by reason of evaporation, transpiration, and percolation. Yield is further subject to variation in rapidity of runoff due to topography, character of the soil, culture and other natural physical features, peculiar to the particular location, and the storage available. The report, therefore, has been divided into three parts, viz: Part I, Introductory, Part II, Yield of Watersheds, and Part III, Rainfall in New England.

Part I. Introductory

The term rainfall is used to include all forms of precipitation.

World Rainfall. In order to afford the membership a basis of appreciation of the relatively uniform rainfall conditions of New England with its freedom from excesses and disastrous periods of drought, except at relatively long intervals, a world map (Figure 1) has been prepared showing the average annual rainfall in general areas for which records are available to the Committee.

The data shown on this map are included in Table 1.

Table 1.—Location of Rainfall Stations on the World Map Shown in Figure 1.

Station No.	Place	Mean Annual Rainfall—Inches	Number of Years of Record
23	Boston, Mass., U.S.A.	43.5	110
75	Washington, D. C., U.S.A.	40.5	85
102	Atlanta, Ga., U.S.A.	48.8	48
127	Galveston, Tex., U.S.A.	47.1	56
134	Amarillo, Tex., U.S.A.	22.0	46

TABLE 1.—(*Continued*)

Station No.	Place	Mean Annual Rainfall—Inches	Number of Years of Record
167	Chicago, Ill., U.S.A.	33.1	84
188	Bismark, N. D., U.S.A.	17.0	51
205	Forks, Wash., U.S.A.	122.3	15
247	Death Valley, Cal., U.S.A.,	1.8	16
273	Salt Lake City, Utah, U.S.A.	16.4	54
331	Greytown, Nicaragua	254.9	6
346	Buenaventura, Columbia	280.6	7
353	La Joya, Peru	0.2	9
360	Punta Galera, Chile	87.5	27
363	Punta Arenas, Chile	15.3	34
369	Buenos Aires, Argentine	37.2	64
373	Asuncion, Paraguay	51.8	29
378	Recife, Brazil	65.0	48
382	Georgetown, British Guiana	88.7	30
406	St. John's, Newfoundland	54.4	52
416	Sitka, Alaska	84.8	64
422	Nome, Alaska	17.4	16
430	Kaanapali, Hawaii	20.2	20
431	Puu Kukui, Hawaii	370.1	7
466A	Cromhamhurst, Australia	80.1	20
471A	Peechywarrina, Australia	4.5	21
498	Tokyo, Japan	59.9	45
505	Honk Kong, China	84.3	37
515	Cherrapunji, India	457.5	62
548	Aden, South Arabia	1.8	40
550	Deim Zubeir, Sudan	45.7	15
567	Cape Town, South Africa	25.3	87
597	Calaba, Nigeria	123.2	24
602	Freetown, Sierra Leone	157.2	46
611	Aswan, Egypt	0.0	20
622	Athens, Greece	15.5	26
672	Saragossa, Spain	12.0	40
715	The Stye, England	176.9	35
738	Liverpool, England	27.9	35
745	Prague, Czecho-Slovakia	18.5	50
768	Kasan, Russia	16.4	54
774	Barnaul, Siberia	11.5	54
788	Obdorsk, Siberia	10.1	27
803	Kola, Lapland	6.9	14
814	Angmagsalik, Greenland	35.6	22
815	Upernivik, Greenland	9.0	47

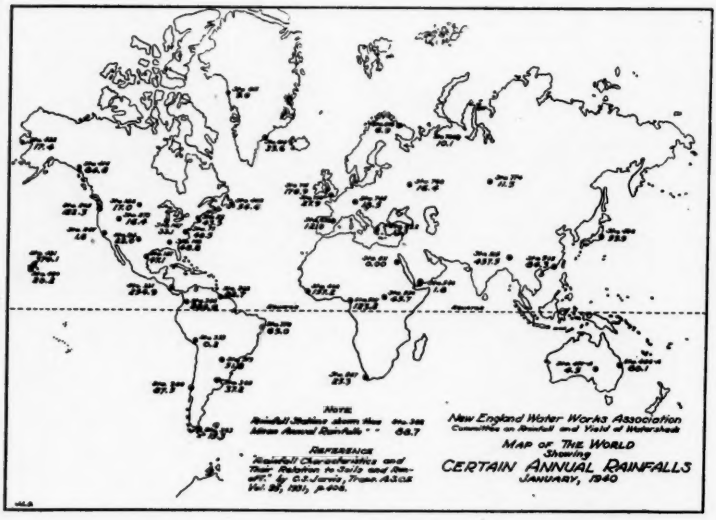


FIGURE 1

In the tropics are found the rainy and dry seasons so frequently mentioned, but even in the tropics the characteristics of rainfall may vary within relatively short distances. One of two stations that receive the same annual precipitation may obtain practically all of its rainfall within a few months, while at the other station the precipitation may occur in nearly equal monthly increments, and two stations having the same monthly distribution of rainfall may vary greatly in the total amount of precipitation for the year. Uneven distribution may be seen also in comparing the rainfall for example in Boston, Mass., and Eureka, Cal., which have almost equal amounts of rainfall annually. In Boston, this rainfall occurs with remarkable uniformity throughout the year, while in Eureka the rainfall distribution is marked by high winter precipitation and low summer rainfall. A comparison may be made of the distribution of rainfall at the widely separated points of Galveston, Tex., and Burlington, Vt., for which the distribution curves are quite similar, but where Burlington receives only about two-thirds as much rain as Galveston.

The subject of rainfall distribution with relation to the climates of the several continents is discussed at length by W. G. Kendrew in "Climates of the Continents", a book published by the Oxford University Press in 1922. Kendrew divides types of rainfall distribution into six classifications as follows:

1. The equatorial type occurring within a few degrees of latitude from the equator has two maxima, which occur about when the sun is directly overhead, but this type of rainfall has no pronounced dry season between maxima.

2. Further from the equator the maxima approach each other, and the winter months become a pronounced dry season. Near the outer limits of the tropical zone, the maxima coalesce, and a longer dry season results.

3. The monsoon type of rainfall occurs inside and outside the tropics, especially on the easterly coasts of the continents and is characterized by a pronounced maximum in summer and a long dry season.

4. The Mediterranean type of rainfall occurs in the six winter months with a single maximum concentrated in November or December or with two maxima occurring in the spring and fall. The summers are quite rainless.

5. In the temperate latitudes, the continental interiors experience most of the rainfall in summer but are without rainless winters. The periodicity of the maxima is not as marked as in the Mediterranean and monsoon types of rainfall.

6. The westerly coasts of the continents in temperate latitudes receive abundant rain in all seasons, but they experience maxima in the autumn or winter months.

Table 2, based for the most part on information included in a paper by C. S. Jarvis entitled "Rainfall Characteristics and Their Relation to Soils and Runoff", published in the Transactions of the American Society of Civil Engineers, Volume 95, 1931, shows the monthly variation in rainfall for stations representing the maximum and minimum for the continents.

New England Rainfall. The studies of Kendrew divide the North American continent into various zones where the rainfalls have certain common characteristics as to seasonal distribution. Accord-

TABLE 2.—MAXIMUM AND MINIMUM RAINFALLS IN INCHES FOR THE CONTINENTS

Station No.	Place	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
Asia including European Russia														
515	Cherrapunji, India	.67	2.31	10.57	31.86*	50.85	103.6	107.4	81.47	49.41	16.78	2.27	0.26	457.5*
548	Aden, Arabia	.32	.19	.48	.18	.12	.06	.03	.12	.15	.09	.08	.11	1.8
Africa														
(1)	Debundja, Kamerun	8.0	10.9	17.1	17.3	24.8	59.7	64.4	57.7	65.2	45.2	26.6	15.1	412.2
602	Freetown, Sierra Leone	0.41	0.30	1.16	4.06	11.47	20.04	35.58	36.57	28.48	12.62	5.12	1.42	157.2
611	Aswan, Egypt	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Australia and New Zealand														
(2)	Harvey Creek	30.9	22.2	32.2	22.2	13.2	8.0	4.2	5.4	3.7	3.8	8.1	11.7	165.6
466a	Crohamhurst	13.56	15.85	13.77	4.99	5.00	4.40	2.88	2.41	2.14	3.69	4.50	6.92	80.1
471a	Peechywarrina	0.74	0.54	0.31	0.40	0.39	0.60	0.16	0.25	0.21	0.16	0.37	0.42	4.5
Europe														
715	The Stye, England	No monthly data												176.9
672	Saragosa, Spain	0.79	0.75	1.02	1.18	1.66	1.06	0.71	0.55	0.94	1.38	1.22	0.71	12.0
803	Kola (3), Russia	0.29	0.25	0.21	0.29	0.54	0.74	1.22	1.07	0.94	0.67	0.44	0.28	6.9
South and Central America														
346	Buenaventura, Colombia	17.90	12.33	11.34	25.60	28.40	23.76	22.35	26.66	27.93	34.66	28.08	21.64	280.6
353	LaJoya, Peru	0.01	0.01	T	T	T	T	0.00	T	0.00	0.00	T	T	0.2
North America excluding Mexico														
205	Forks, Wash.	19.49	13.49	14.76	8.37	4.95	3.86	1.27	1.95	5.28	12.34	18.45	18.13	122.3
247	Death Valley, Cal.	0.46	0.31	0.19	0.02	0.07	0.05	0.06	0.04	0.14	0.09	0.27	0.09	1.8
Pacific Islands														
431	Puu Kukui, Hawaii	25.81	22.52	32.61	46.91	26.00	33.51	35.09	28.80	22.20	24.93	38.21	32.48	370.1
430	Kaanapali, Hawaii	4.22	3.24	2.26	1.43	0.93	0.36	0.48	1.22	0.50	0.69	1.55	3.24	20.2

*Corrected in accordance with errata sheet, published in "World Weather Records", Smithsonian Institution, 1927.
(1) Debundja, Kamerun, from W. G. Kendrew "Climates of the Continents".
(2) Harvey Creek, Australia, from W. G. Kendrew "Climates of the Continents".

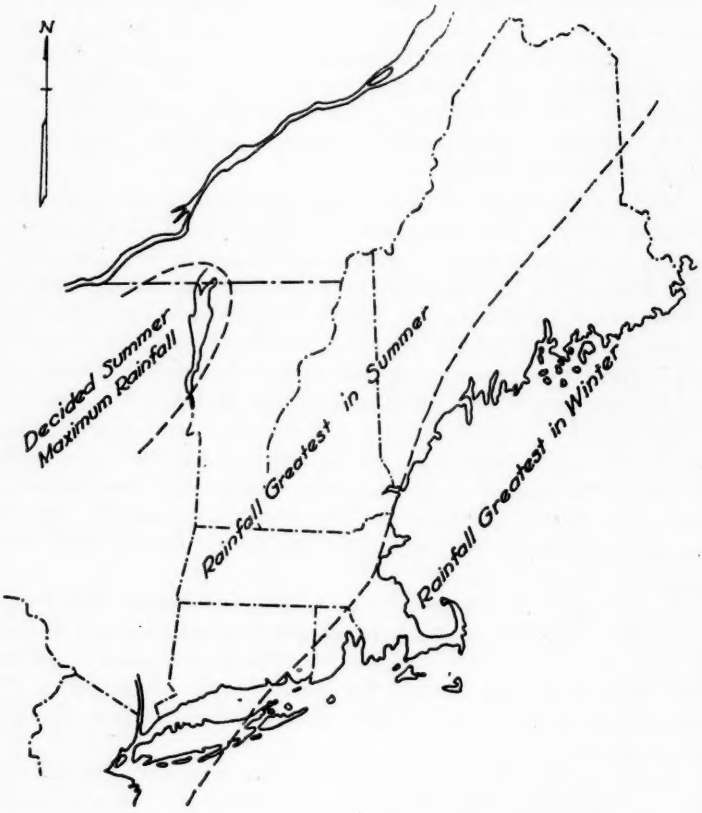
ing to his classification, portions of New England fall within three separate zones. The northwesterly portion of New England, in the Lake Champlain region, is characterized by higher rainfall in summer, while the coastal areas are characterized by somewhat greater winter precipitation. The precipitation in the region lying between these two is fairly uniform throughout the year, with a slight excess in the months of July and August.

In the early paper by Goodnough, published in the Journal for September, 1915, certain long-term records are referred to. These may be divided into three groups: Group I, including Gardner, Me., Cambridge, Boston and New Bedford, Mass., and Providence, R. I., which is characterized by a somewhat greater rainfall in winter; Group II, including Lowell, Waltham, Amherst and Springfield, Mass., which is characterized by somewhat greater rainfall in summer; Group III, including Burlington, Vt., alone of the long term stations, which is characterized by a decided summer maximum. The New England zones herein referred to are shown in Figure 2, and the seasonal distribution of the rainfall at the long-term stations that have been enumerated are shown in Figure 3. The winter and summer maxima, with the exception of Burlington, Vt., do not warrant plotting them on separate diagrams.

In his paper of September, 1915, Mr. Goodnough presented a map of New England showing the geographical distribution of rainfall, and it is thought of sufficient interest to reproduce it in this report (Figure 4), especially as the paper is now out of print. Detailed studies in 1930 showed little change from the lines of equal rainfall drawn in 1915.

Long-term Rainfall. There are but few continuous rainfall records for New England equalling or approaching 100 years in length. The records for ten of these stations,[2] which have been previously mentioned in connection with the distribution of rainfall, have been tabulated by the Committee to 1938 inclusive, together with the departure for each year from the median rainfall. The general monthly average rainfall as represented by these records amounts to 3½ inches per month with an average annual rainfall of approximately 42½

[2]New Bedford from 1814, Boston from 1818, Waltham from 1825, Lowell from 1826, Providence from 1832, Amherst from 1836, Cambridge from 1841, Springfield from 1848, Gardner from 1837, and Burlington from 1838.



New England Water Works Association
Committee on Rainfall and Yield of Watersheds

TYPES OF RAINFALL IN NEW ENGLAND

AT LONG TERM STATIONS TO 1928

After
W. G. Kendrew "The Climates of the Continents"
Figure 108

FIGURE 2

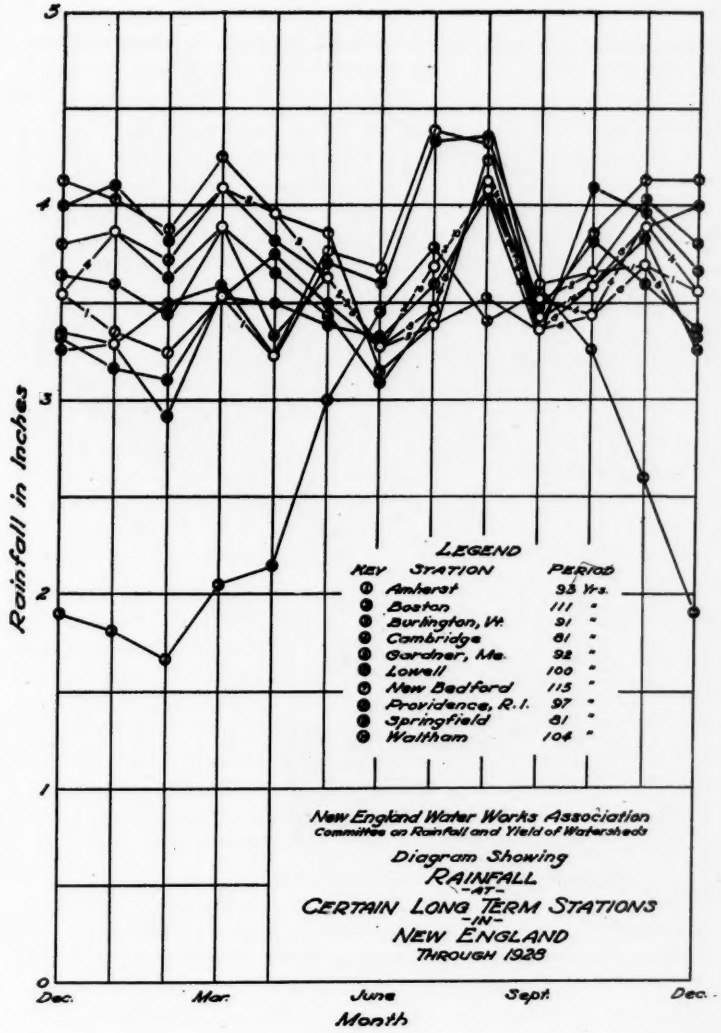

Rainfall in Inches
5
4
3
2
1
0
Dec.
Mar.
June
Sept.
Dec.
Month
LEGEND
KEY STATION PERIOD
1 Amherst 93 Yrs.
2 Boston 111 "
3 Burlington, Vt. 91 "
4 Cambridge 81 "
5 Gardner, Me. 92 "
6 Lowell 100 "
7 New Bedford 115 "
8 Providence, R.I. 97 "
9 Springfield 81 "
10 Waltham 104 "
New England Water Works Association
Committee on Rainfall and Yield of Watersheds
Diagram Showing
RAINFALL
-AT-
CERTAIN LONG TERM STATIONS
-IN-
NEW ENGLAND
THROUGH 1928

FIGURE 3

QUEBEC
NEW BRUNSWICK
MAINE
VERMONT
NEW HAMPSHIRE
NEW YORK
ATLANTIC OCEAN
MASSACHUSETTS
CONNECTICUT
RHODE ISLAND
MAP OF
NEW ENGLAND
SHOWING
LINES OF EQUAL RAINFALL
AND
RAINFALL STATIONS
JAN. 1914.
NOTE
BASED MAINLY ON RECORDS COVERING A PERIOD OF 40 YEARS
SCALE OF MILES
0 10 20 30 40 50

FIGURE 4

inches. The records show that monthly rainfalls have been experienced in amounts as great as 18¾ inches at one of the ten stations enumerated, and it is certain that at other stations with shorter records greater maximum monthly rainfalls have been experienced within the periods of record. As a contrast to these maxima, certain stations have recorded no rainfall at all during certain months. The annual rainfall in New England has varied from as little as 22.62 inches at Burlington, Vt., in 1914, to as much as 67.72 inches at Boston, Mass., in 1863. At certain stations of high elevation, the precipitation has been considerably greater in certain years, and it is possible that the annual rainfall in certain localities may have been less than the minimum shown for Burlington, Vt.

Storm Rainfall. Single storms of record have exceeded 15 inches over widespread areas, notably that of September, 1938, in the Ware River Valley in Massachusetts, where the rainfall for a five-day period exceeded 17 inches at one station, and it is probable that this rainfall exceeded that of any month on record for New England stations, except those of very high altitude. A paper by X. H. Goodnough, published in the June, 1928, Journal of the Association, following the great storm of November 3 and 4, 1927, which occasioned the so-called Vermont flood, reviews other great storms: the storm of October 3 and 4, 1869, central over northwestern Connecticut, in which the maximum rainfall equalled 12.35 inches; the storm of February 10 to 14, 1886, centered near Providence, R. I., in which the rainfall exceeded 9 inches; the storm of October 12 to 14, 1895, central over the territory southwest of Boston, Mass., in which the rainfall exceeded 8 inches; and the storm of November 3 and 4, 1927, in which more than 9 inches rain fell in the Green Mountains of Vermont and rainfalls exceeding 8 inches and 9 inches occurred in the southwesterly portion of Rhode Island and in the area east and southeast of Worcester, Mass. A detailed description of these storms is given by Goodnough and references are made to certain other storms that caused freshets in various localities.

Droughts. Droughts have occurred with sufficient frequency to warrant water works officials seriously considering extended periods of low rainfall and their effects upon the amount of water available to the communities under their charge. This is especially important as

many of the water supplies in New England have insufficient storage to equalize variations in yield over a period of more than two to three years. The progressive average rainfall for three-year periods is shown on a plot published in the September, 1915, paper by X. H. Goodnough and brought up to date by G. V. White in a paper by Francis H. Kingsbury on "Public Surface Water Supplies in Massachusetts" published in the Journal of the Association for March, 1939. A similar diagram showing the average annual precipitation at eleven selected stations in New England is shown in a paper by Arthur T. Safford, a member of the Committee and published in the April, 1939, Journal of the Boston Society of Civil Engineers. In this paper, Mr. Safford lists periods of low rainfall in the order of their dryness for periods of from one to twenty-five years.

Prior to 1908, those concerned with computations of the yield of drainage areas based them upon the dry period from 1879 to 1883, since few records of yield were available prior to those dates. After records for the dry period that began in June, 1908, become available, it was apparent that this later period was drier than the earlier one. The period extended with less than normal annual rainfall through the year 1918. The years 1924, 1930, 1935 and 1939, too, had rainfalls in general far below normal.

Part II. Yield of Watersheds

For a given rainfall, a particular watershed may be expected to yield as much water in the future, as it did in the past, subject to such changes as may be brought about in the character of the watershed by the works of man or by nature, and subject to such cyclical variations as our existing records have not disclosed. In all estimates of yield, certain basic assumptions must be made for drainage areas for which records of yield are available and additional assumptions where records of yield are not available.

The primary records necessary for yield calculations are those showing the flow of the stream called the *runoff*. With concurrent records of the amount added to or drawn from storage on the watershed, the flow measurements may first be corrected to represent the natural flow or yield.

Available records of yield show variations influenced to a con-

siderable extent by the proportions of water surface in lakes, ponds, streams, reservoirs, or swamp areas. It has been found necessary, in order to make such records useful, to subdivide the estimated total yield into land yield and yield of water surface. In order to do this, it is necessary, in the absence of continuous records of evaporation, to assume values for the evaporation from water surfaces.

Evaporation. Experiments to determine the amount and seasonal variation of evaporation conducted by Desmond Fitzgerald are shown in his paper on "Rainfall, Flow of Streams, and Storage" which was published in Transactions of the American Society of Civil Engineers, September 1892, page 275.

The records of evaporation made by the Water Bureau of the Metropolitan District Commission of Hartford, Conn., at Nepaug Reservoir and Reservoir No. 1, extending over the period from 1917 to 1939 inclusive, have been submitted to the Committee, and a comparison of the averages of these observations with those of Fitzgerald is shown in Table 3.

TABLE 3.—COMPARISON OF EVAPORATION MEASUREMENTS IN INCHES AT BOSTON AND HARTFORD

Month	Fitzgerald (1876-1892) Chestnut Hill Reservoir	Water Bureau of the Metropolitan District Commission, Hartford, Conn. (1917 to 1939) Nepaug Reservoir	No. 1
Jan.	0.96	—	—
Feb.	1.05	—	—
Mar.	1.70	—	—
Apr.	2.97	(2)* 1.49	(3) 3.81
May	4.46	(12) 3.33	(17) 4.96
Jun.	5.54	(18) 4.17	(20) 5.51
Jul.	5.98	(20) 5.28	(21) 6.03
Aug.	5.50	(21) 4.85	(20) 5.39
Sep.	4.12	(21) 3.53	(18) 4.03
Oct.	3.16	(13) 2.48	(12) 3.18
Nov.	2.25	(3) 2.45	(3) 1.78
Dec.	1.51	—	—
Total	39.20	27.58	34.39
April to November Inclusive	33.98	—	—

*Figures in parentheses indicate number of months of record.

Yield of Water Surface. The yield of water surface is estimated by subtracting the evaporation from the rainfall. In the absence of accurate, continuous records of evaporation, the Committee is of the opinion that the use of Fitzgerald's evaporation values as a standard for Massachusetts at least is within the degree of accuracy of most yield computations.

Under the heading of water surface, it has been customary to include a portion of the swamp area. The Committee on Yield of Drainage Areas, in its report published in 1914, recommended that 40 per cent of undrained swamps and 30 per cent of carefully drained swamps be included as water surface.

The annual yield of water surface of Wachusetts Reservoir from January, 1897, to December, 1938, using Fitzgerald's evaporation values, averages about 321,000 gallons per day per square miles, which will later be shown to be approximately one-quarter of the yield of land surface. During periods of droughts, however, the yield of water surface is negative and varies from nearly minus 1,000,000 gallons per day per square mile of water surface for a single month to about minus 10,000 gallons per day per square mile of water surface for extended dry periods such as the period from June, 1908, to November, 1915, inclusive. These variations for dry periods of various duration are shown in Table 4.

Land Yield. In estimating the land yield, the yield of water surface is deducted from the total yield obtained by correcting the runoff for gain or loss in storage. The estimated land yields of various watersheds in New England were included in the Report of the Committee on Yield of Drainage Areas in 1914 and as many of these records as possible have been brought up to date and are included herewith. The earlier report considered data available through July, 1914, but the published tables for the most part only extended through 1912. It was apparent from these records that the dry period beginning with June, 1908, exceeded in severity earlier droughts upon which the safe yield of watersheds had previously been computed. Previous to the work of the 1914 committee the period from 1879 to 1883 had been considered the driest on record for New England, although precipitation records indicate that a still drier period occurred prior to the year 1850. The period beginning with June, 1908,

TABLE 4.—ESTIMATED YIELD OF WATER SURFACE OF WACHUSETT RESERVOIR USING FITZGERALD'S EVAPORATION VALUES.
JANUARY, 1897, TO DECEMBER, 1938.

Number of Consecutive Months	Driest Consecutive Months on the Basis of Yield from Land Surfaces Inclusive Dates From Mo.	Yr.	To Mo.	Yr.	Average Yield of Water Surface—1000 gpd per sq. mi.
1	—	—	Oct.	1910	—987
2	Sep.	1910	Oct.	1910	—860
3	Sep.	1914	Nov.	1914	—865
4	Sep.	1914	Dec.	1914	—306
5	Aug.	1914	Dec.	1914	—358
6	Jul.	1910	Dec.	1910	—601
7	Jul.	1924	Jan.	1925	—192
8	Jun.	1924	Jan.	1925	—482
9	Jun.	1910	Feb.	1911	—268
10	May	1910	Feb.	1911	—374
11	Apr.	1930	Feb.	1931	—226
12	Sep.	1910	Aug.	1911	—244
13	Sep.	1910	Sep.	1911	—273
14	Aug.	1910	Sep.	1911	—320
15	Jul.	1910	Sep.	1911	—468
16	Jul.	1910	Oct.	1911	—364
17	Aug.	1929	Dec.	1930	—259
18	Aug.	1929	Jan.	1931	—166
19	Jul.	1929	Jan.	1931	—310
20	Jul.	1929	Feb.	1931	—249
21	Jun.	1929	Feb.	1931	—304
22	May	1929	Feb.	1931	—286
23	Apr.	1910	Feb.	1912	—182
24	Jun.	1929	May	1931	—174
25	Sep.	1909	Sep.	1911	—161
26	Aug.	1909	Sep.	1911	—197
27	Jul.	1909	Sep.	1911	—226
28	Jul.	1909	Oct.	1911	—176
29	Jun.	1909	Oct.	1911	—219
30	Jun.	1909	Nov.	1911	—176
31	Jun.	1929	Dec.	1931	—187
32	Jun.	1909	Jan.	1912	—109
33	Jun.	1909	Feb.	1912	— 82
34	May	1909	Feb.	1912	—110
35	Apr.	1909	Feb.	1912	— 62

TABLE 4.—(*Continued*)

Number of Consecutive Months	Driest Consecutive Months on the Basis of Yield from Land Surfaces Inclusive Dates From Mo.	Yr.	To Mo.	Yr.	Average Yield of Water Surface— 1000 gpd per sq. mi.
36	Apr.	1910	Mar.	1913	— 24
37	Sep.	1908	Sep.	1911	— 89
38	Aug.	1908	Sep.	1911	— 72
39	Jul.	1908	Sep.	1911	— 87
40	Jun.	1908	Sep.	1911	—160
41	Jun.	1908	Oct.	1911	—127
42	Jun.	1908	Nov.	1911	— 98
43	Jun.	1908	Dec.	1911	— 76
44	Jun.	1908	Jan.	1912	— 53
45	Jun.	1908	Feb.	1912	— 35
46	Apr.	1910	Jan.	1914	— 60
47	Apr.	1910	Feb.	1914	— 28
48	Mar.	1908	Feb.	1912	— 13
49	Sep.	1909	Sep.	1913	— 35
50	Aug.	1909	Sep.	1913	— 67
51	Jul.	1909	Sep.	1913	— 74
52	Jul.	1909	Oct.	1913	— 41
53	Jul.	1909	Nov.	1913	— 37
54	Jun.	1909	Nov.	1913	— 62
55	Jun.	1909	Dec.	1913	— 49
56	Jun.	1909	Jan.	1914	— 23
57	Jun.	1909	Feb.,	1914	+ 2
58	May	1909	Feb.	1914	— 15
59	Apr.	1908	Feb.	1913	— 2
60	Jul.	1910	Jun.	1915	— 46
90	Jun.	1908	Nov.	1915	— 9
126	Jun.	1908	Nov.	1918	+ 12
504	Jan.	1897	Dec.	1938	+321

continues to be the driest long period for which records are available. Estimates based on the yield of the Wachusett catchment area show that some of the shorter dry periods on record have occurred since 1914.

Comparative estimates of the land yield of the Wachusett catchment area that make use of recent data are shown in Table 5 and in Figure 5, a diagram presented with the first progress report of the Committee in 1939.

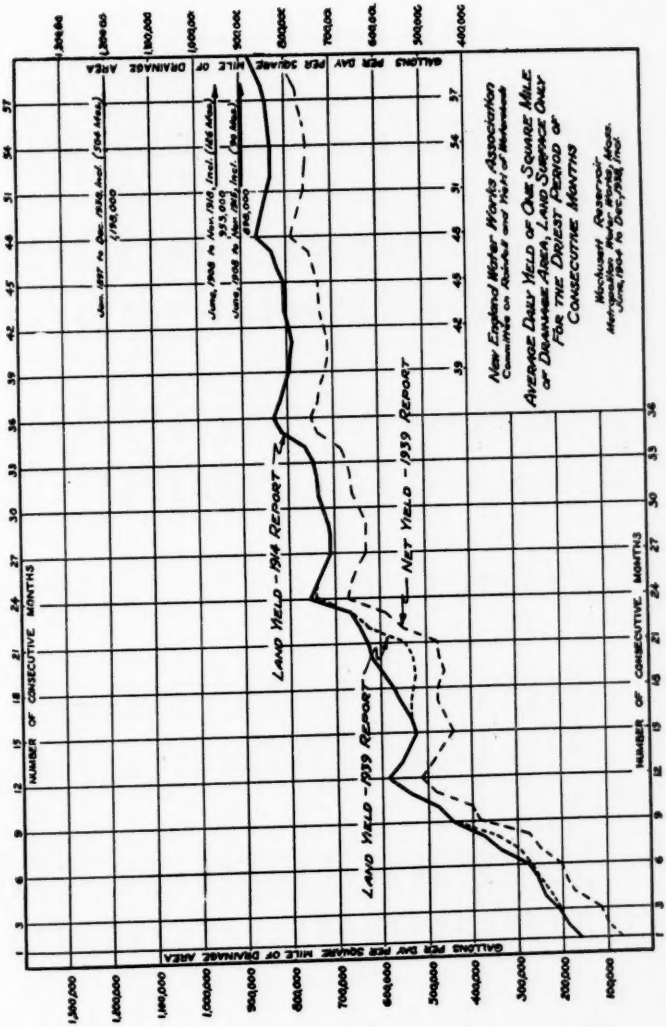

New England Water Works Association
Committee on Additions and Hydrology
Average Daily Yield of One Square Mile of Drainage Area, Land Surface Only for the Driest Period of Consecutive Months
Merrimack Reservoir
Nashua, N.H.
Nov. 1907 to Dec. 1908, Incl. (1266 Miles)
1,785,000
June, 1908 to Nov. 1908, Incl. (105 Miles)
933,000
June, 1908 to Nov. 1908, Incl. (94 Miles)
876,000
LAND YIELD - 1914 REPORT
LAND YIELD - 1939 REPORT
NET YIELD - 1939 REPORT
GALLONS PER DAY PER SQUARE MILE OF DRAINAGE AREA
NUMBER OF CONSECUTIVE MONTHS
DRAINAGE AREA

FIGURE 5

TABLE 5.—PERIODS OF LOW YIELD ON LAND SURFACE FOR THE WACHUSETT CATCHMENT AREA.

No. of Consecutive Months	1914 Report					Present Report					Deficiency in 1939 under 1914 Data— 1000 gpd per sq. mi.
	Inclusive Dates				Average Yield— 1000 gpd per sq. mi.	Inclusive Dates				Average Yield— 1000 gpd per sq. mi.	
	From		To			From		To			
	Mo.	Yr.	Mo.	Yr.		Mo.	Yr.	Mo.	Yr.		
1	—	—	Oct.	1910	158	—	—	Oct.	1910	158	0
2	Sep.	1910	Oct.	1910	190	Sep.	1910	Oct.	1910	190	0
3	Jul.	1913	Sep.	1913	205	Sep.	1914	Nov.	1914	204	1
4	Jul.	1910	Oct.	1910	240	Sep.	1914	Dec.	1914	226	14
5	Jul.	1910	Nov.	1910	251	Aug.	1914	Dec.	1914	249	2
6	Jul.	1910	Dec.	1910	274	Jul.	1910	Dec.	1910	274	0
7	Jul.	1908	Jan.	1909	334	Jul.	1924	Jan.	1925	294	40
8	Jun.	1908	Jan.	1909	376	Jun.	1924	Jan.	1925	350	26
9	Jun.	1910	Feb.	1911	442	May	1910	Feb.	1911	442	0
10	May	1910	Feb.	1911	477	Jun.	1910	Feb.	1911	447	0
11	Apr.	1910	Feb.	1911	536	Apr.	1930	Feb.	1931	533	3
12	Sep.	1910	Aug.	1911	582	Sep.	1910	Aug.	1911	582	0
13	Sep.	1910	Sep.	1911	557	Sep.	1910	Sep.	1911	557	0
14	Aug.	1910	Sep.	1911	537	Aug.	1910	Sep.	1911	537	0
15	Jul.	1910	Sep.	1911	521	Jul.	1910	Sep.	1911	521	0
16	Jul.	1910	Oct.	1911	532	Jul.	1910	Oct.	1911	532	0
17	Jun.	1910	Oct.	1911	556	Aug.	1929	Dec.	1930	532	24
18	May	1910	Oct.	1911	570	Aug.	1929	Jan.	1931	527	43
19*	May	1910	Oct.	1911	594	Jul.	1929	Jan.	1931	523	71
20*	Jul.	1910	Feb.	1912	618	Jul.	1929	Feb.	1931	530	88
21*	May	1910	Jan.	1912	627	Jun.	1929	Feb.	1931	542	85
22*	May	1910	Feb.	1912	642	May	1929	Feb.	1931	625	17
23*	Apr.	1910	Feb.	1912	663	Apr.	1910	Feb.	1912	663	0
24*	Apr.	1910	Mar.	1912	756	Jun.	1929	May	1931	743	13
25*	Sep.	1909	Sep.	1911	740	Sep.	1909	Sep.	1911	740	0

TABLE 5.—(*Continued*)

No. of Consecutive Months	1914 Report Inclusive Dates From Mo.	From Yr.	To Mo.	To Yr.	1914 Report Average Yield—1000 gpd per sq. mi.	Present Report Inclusive Dates From Mo.	From Yr.	To Mo.	To Yr.	Present Report Average Yield—1000 gpd per sq. mi.	Deficiency in 1939 under 1914 Data—1000 gpd per sq. mi.
26*	Aug.	1909	Sep.	1911	723	Aug.	1909	Sep.	1911	723	0
27*	Jul.	1909	Sep.	1911	709	Jul.	1909	Sep.	1911	709	0
28*	Jul.	1909	Oct.	1911	708	Jul.	1909	Oct.	1911	708	0
29*	Jun.	1909	Oct.	1911	712	Jun.	1909	Oct.	1911	712	0
30*	Jun.	1909	Nov.	1911	723	Jun.	1909	Nov.	1911	723	0
31*	Jul.	1909	Jan.	1912	734	Jun.	1929	Dec.	1931	733	1
32*	Jun.	1909	Jan.	1912	737	Jun.	1909	Jan.	1912	757	0
33*	Jun.	1909	Feb.	1912	743	Jun.	1909	Feb.	1912	743	0
34*	May	1909	Feb.	1912	763	May	1909	Feb.	1912	763	0
35*	Apr.	1909	Feb.	1912	812	Apr.	1909	Feb.	1912	812	0
36*	Apr.	1910	Mar.	1913	826	Apr.	1910	Mar.	1913	826	0
37*	Sep.	1908	Sep.	1911	816	Sep.	1908	Sep.	1911	816	0
38*	Aug.	1908	Sep.	1911	806	Aug.	1908	Sep.	1911	806	0
39*	Jul.	1908	Sep.	1911	794	Jul.	1908	Sep.	1911	794	0
40*	Jun.	1908	Sep.	1911	791	Jun.	1908	Sep.	1911	791	0
41*	Jun.	1908	Oct.	1911	788	Jun.	1908	Oct.	1911	788	0
42*	Jun.	1908	Nov.	1911	794	Jun.	1908	Nov.	1911	794	0
43*	Jun.	1908	Dec.	1911	801	Jun.	1908	Dec.	1911	801	0
44*	Jun.	1908	Jan.	1912	801	Jun.	1908	Jan.	1912	801	0
45*	Jun.	1908	Feb.	1912	804	Jun.	1908	Feb.	1912	804	0
46*	Apr.	1910	Jan.	1914	817	Apr.	1910	Jan.	1914	817	0
47*	Apr.	1910	Feb.	1914	824	Apr.	1910	Feb.	1914	824	0
48*	Mar.	1908	Feb.	1912	863	Mar.	1908	Feb.	1912	863	0
49*	Sep.	1909	Sep.	1913	860	Sep.	1909	Sep.	1913	860	0
50*	Aug.	1909	Sep.	1913	848	Aug.	1909	Sep.	1913	848	0

TABLE 5.—(*Continued*)

No. of Consecutive Months	1914 Report					Present Report					Deficiency in 1939 under 1914 Data— 1000 gpd per sq. mi.
	Inclusive Dates				Average Yield— 1000 gpd per sq. mi.	Inclusive Dates				Average Yield— 1000 gpd per sq. mi.	
	From		To			From		To			
	Mo.	Yr.	Mo.	Yr.		Mo.	Yr.	Mo.	Yr.		
51*	Jul.	1909	Sep.	1913	839	Jul.	1909	Sep.	1913	839	0
52*	Jul.	1909	Oct.	1913	834	Jul.	1909	Oct.	1913	834	0
53*	Jul.	1909	Nov.	1913	832	Jul.	1909	Nov.	1913	832	0
54*	Jun.	1909	Nov.	1913	831	Jun.	1909	Nov.	1913	831	0
55*	Jun.	1909	Dec.	1913	834	Jun.	1909	Dec.	1913	834	0
56*	Jun.	1909	Jan.	1914	836	Jun.	1909	Jan.	1914	836	0
57*	Jun.	1909	Feb.	1914	842	Jun.	1909	Feb.	1914	842	0
58*	May	1909	Feb.	1914	852	May	1909	Feb.	1914	852	0
59*	Apr.	1908	Feb.	1913	870	Apr.	1908	Feb.	1913	870	0
60*	Oct.	1908	Sep.	1913	893	Jul.	1910	Jun.	1915	882	11
90						Jun.	1908	Nov.	1915	898	
126						Jun.	1908	Nov.	1918	953	
504						Jan.	1897	Dec.	1938	1,198	

*Computed in 1939 from data available at time of compilation of 1914 Reports (42 years).

The ultimate value of records and computation of yield of watersheds is to permit the computation of the safe yield of proposed water supply projects. The safe yield, sometimes called the dependable yield, may be defined as that quantity of water that may be drawn continuously from a given water supply development. Since natural watersheds include certain percentages of water surface and additional water surface is introduced by the construction of reservoirs, that yield upon which the safe yield is estimated is a summation of the yields of the water surface and land area. These combined yields in themselves vary from month to month and from year to year, requiring the construction of reservoirs or the use of natural lakes and ponds to equalize these variations in total yield.

Relation of Rainfall to Land Yield. The 1914 Report of the Committee on the Yield of Drainage Areas discussed at considerable length the relation of precipitation to yield, showing by tabular arrangements a comparison of the runoff in years of high and low rainfall and the per cent of precipitation running off in different portions of the year. It seems unnecessary to duplicate these studies in the present report. Now that relatively long records of yield are available, however, it is interesting again to consider the relation between annual rainfall and land yield. In order that ground storage may be partially eliminated from consideration, the climatological year beginning with October 1 has been used rather than the calendar year Two diagrams are presented herewith, one (Figure 6) showing the relation between the land yield and the rainfall for the climatological years since October 1, 1908, and a similar diagram (Figure 7) showing the relation between the three-year progressive totals of land yield and rainfall for the same period, the three-year progressive totals being selected because the critical periods for the safe yield for most of the smaller water supplies do not exceed three years.

It will be seen from these diagrams that there is a fairly good correlation between rainfall and land yield. A casual examination might indicate that with an annual precipitation of 20 inches per year no yield should be expected. But further consideration must convince us that such a conclusion would be erroneous, since with low rainfalls a gradual reduction in the land yield would take place until the amounts lost by evaporation, transpiration, and percolation to

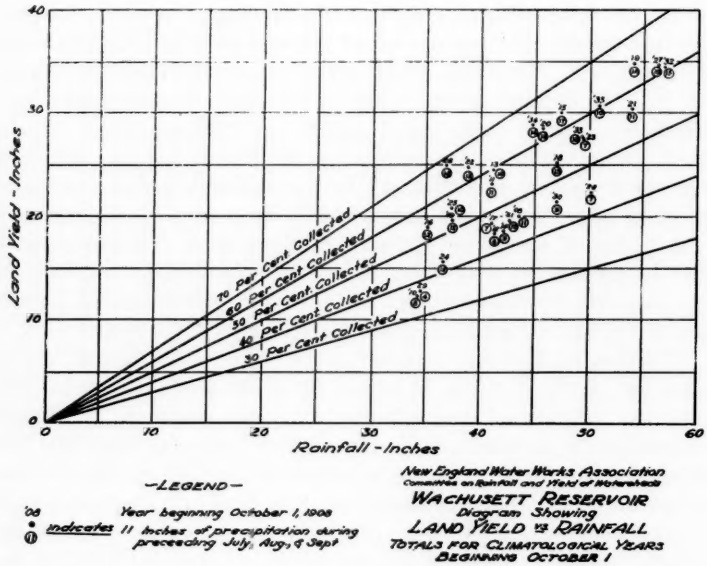

Land Yield - Inches
Rainfall - Inches
70 Per Cent Collected
60 Per Cent Collected
50 Per Cent Collected
40 Per Cent Collected
30 Per Cent Collected
—LEGEND—
'08 Year beginning October 1, 1908
indicates 11 inches of precipitation during preceding July, Aug. & Sept.
New England Water Works Association
Committee on Rainfall and Yield of Watersheds
WACHUSETT RESERVOIR
Diagram Showing
LAND YIELD vs RAINFALL
TOTALS FOR CLIMATOLOGICAL YEARS
BEGINNING OCTOBER 1

FIGURE 6

subterranean depths should result in a total lack of stream flow. The diagrams do show, however, that with high rainfalls a more or less proportionate increase in land may be expected and that, while 40 per cent of the rainfall may be collected in drier periods, as much as 60 per cent may be collected in wetter ones.

Consideration of Figure 7 showing the three-year progressive totals, indicates that for such periods an increase in average rainfall above 40 inches per year would result in 90 per cent of this increase being collected as yield. This is a generalization covering wet as well as dry periods and would be misleading if applied to the critical periods upon which safe yields are estimated.

The suggestion has been made that the Committee determine the effect upon the total yield of evaporations greater or less than Fitzgerald's values which are commonly used in New England, and also the effect upon the yield of variations in the proportion of water

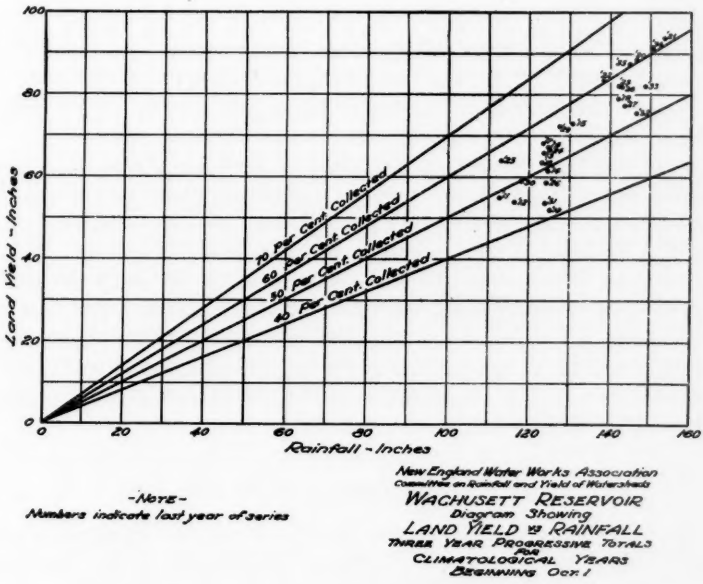


FIGURE 7

surface. In general, it may be stated that for periods in excess of two years the safe yield of a watershed is reduced in proportion to the amount of water surface. This should be apparent when it is recognized that the average annual evaporation and rainfall, especially during extended dry periods, are nearly equal. This fact has led some engineers to discount the water surface entirely in computations of safe yield. For shorter periods, however, the proportion of water surface has a much greater effect in reducing the safe yield. These conclusions are based upon computations using Fitzgerald's figures.

Storage Requirements. The amount of storage required to develop the maximum safe yield will increase with the magnitude of the accumulated deficiency in yield. The degree to which it is economical to develop a watershed will be determined to a large extent by the availability of other drainage areas and of the natural topographical features that affect the possible size of reservoir. More-

over, it may not be feasible or advisable to use all of the storage developed because of the long periods during which the bottom of the reservoir might be exposed and because of the possibility that the water at the bottom of the reservoir might not be of acceptable quality.

In the 1914 Report on the Yield of Drainage Areas, tables and curves were prepared for several watersheds showing the relation between the safe yield to be expected from certain watersheds with varying amounts of storage and percentages of water surface. Making use of the information that has become available since the earlier report, the following diagrams show for the Wachusett and Sudbury watersheds a comparison of the earlier curves with those developed on the basis of all information prior to 1940 (Figures 8 to 11). The Committee is giving consideration to the preparation of similar curves for additional drainage areas.

Part III. Rainfall in New England

A compilation of the records of rainfall in New England is being prepared by the Division of Sanitary Engineering of the Massachusetts Department of Public Health under the direction of George V.

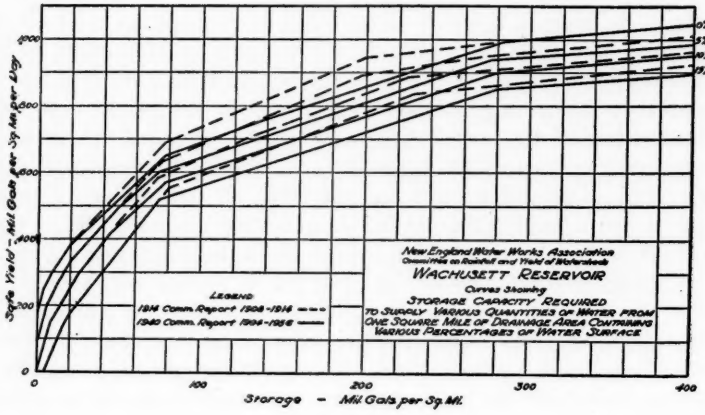


Figure 8

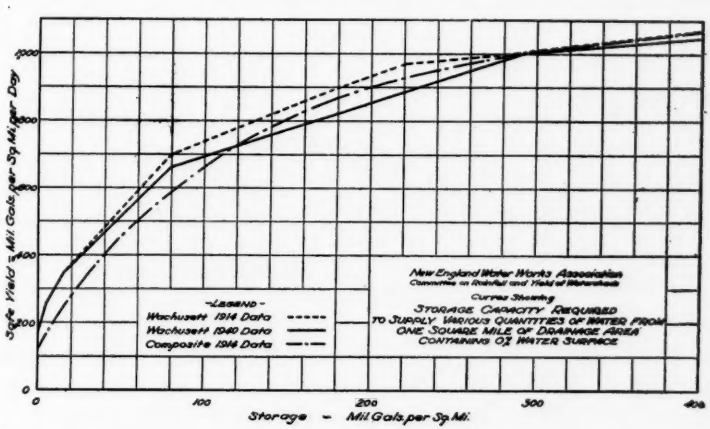

Safe Yield - Mil Gals per Sq Mi per Day
Storage - Mil Gals per Sq Mi
-LEGEND-
Wachusett 1914 Data
Wachusett 1940 Data
Composite 1914 Data
New England Water Works Association
Committee on Rainfall and Yield of Watersheds
Curves Showing
STORAGE CAPACITY REQUIRED
TO SUPPLY VARIOUS QUANTITIES OF WATER FROM
ONE SQUARE MILE OF DRAINAGE AREA
CONTAINING 0% WATER SURFACE

FIGURE 9

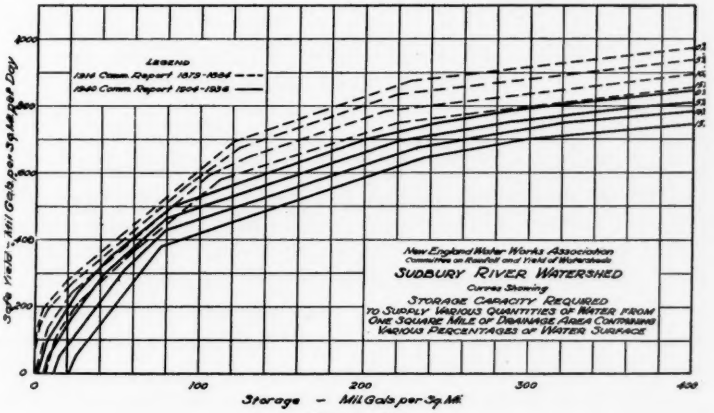

Safe Yield - Mil Gals per Sq Mi per Day
Storage - Mil Gals per Sq Mi
LEGEND
1914 Comm. Report 1873-1936
1940 Comm. Report 1906-1936
New England Water Works Association
Committee on Rainfall and Yield of Watersheds
SUDBURY RIVER WATERSHED
Curves Showing
STORAGE CAPACITY REQUIRED
TO SUPPLY VARIOUS QUANTITIES OF WATER FROM
ONE SQUARE MILE OF DRAINAGE AREA CONTAINING
VARIOUS PERCENTAGES OF WATER SURFACE

FIGURE 10

White, who has been made a member of this committee.[3]

Your Committee respectfully presents its second progress report, the first having been submitted in September, 1939.

For the Committee,

(Signed) ARTHUR D. WESTON, *Chairman*

[3]EDITOR'S NOTE: As previously stated these records have been published since the report of the Committee was made.

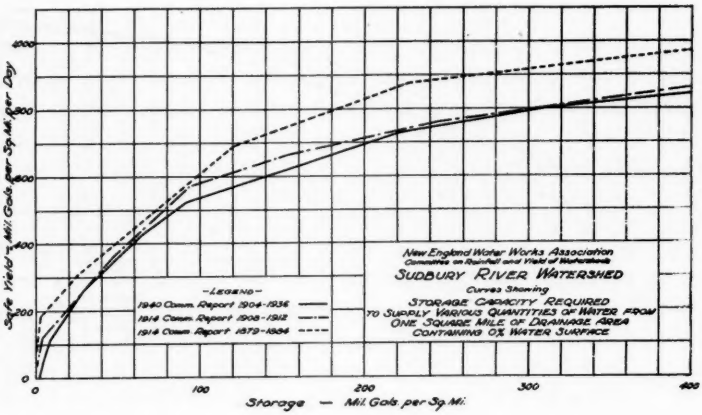


FIGURE 11

THIRD PROGRESS REPORT OF THE COMMITTEE ON RAINFALL AND YIELD OF WATERSHEDS IN NEW ENGLAND

September 25, 1941

To the New England Water Works Association, Gentlemen:

The Committee on Rainfall and Yield of Watersheds in New England appointed in November 1938 has made two progress reports, the first dated September 5, 1939, and the second dated September 24, 1940. The Committee now presents its third progress report.

The first progress report was relatively brief and contained information as to periods of low yield on land surfaces with particular regard to Wachusetts Reservoir.

The second progress report precedes the present one in this issue of the Journal.

During the past year all members of the Committee have been very busy, many in connection with problems brought about by the National Defense program. Nevertheless, a certain amount of additional information has been made available as follows:

a. Additional information relative to the evaporation figures for certain Maine stations, obtained in 1905 to 1909 by H. K. Barrows

when he was connected with the U. S. Geological Survey, relating to Soldier's Pond, Millinocket, Lewiston and Mooselookmeguntic Lake. (See Water Supply Paper No. 279, U. S. Geological Survey, Kenobscot River Basin, pp. 113-129.)

b. Storage yield relations for the following additional watersheds:

1. Tucker Brook watershed, formerly Manhan River, 1914 (Figure 12).
2. Tillotson Brook (Figure 13).
3. Nepaug Reservoir (Figure 14).
4. Abbott Run (Figure 15).
5. Croton Watershed (Figure 16).
6. Assembly and composite curve based on 1940 data (Figure 17).
7. Composite curve compared with 1914 Composite curve (Figure 18).

c. Previously unpublished records of the flow of five streams in Massachusetts prepared by George V. White, Assistant Sanitary Engineer, Department of Public Health, August 1941.

1. Charles River at Charles River Village.

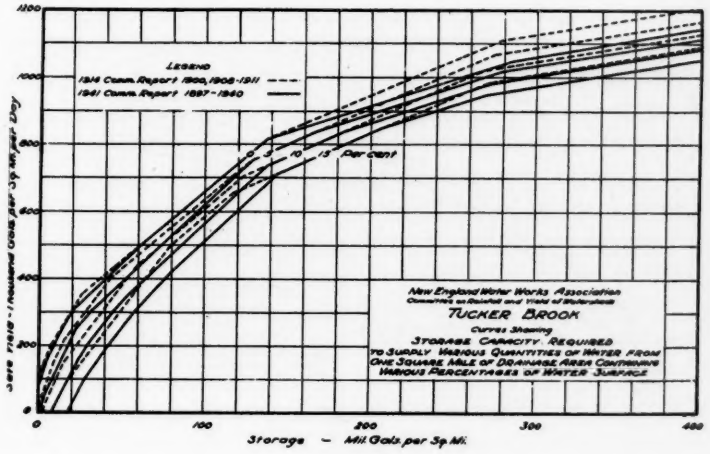

Legend
1914 Comm. Report 1900, 1908-1911
1941 Comm. Report 1897-1940
0 5 10 15 Per cent
New England Water Works Association
TUCKER BROOK
Curves Showing
STORAGE CAPACITY REQUIRED TO SUPPLY VARIOUS QUANTITIES OF WATER FROM ONE SQUARE MILE OF DRAINAGE AREA CONTAINING VARIOUS PERCENTAGES OF WATER SURFACE
Storage — Mil. Gals. per Sq. Mi.

FIGURE 12

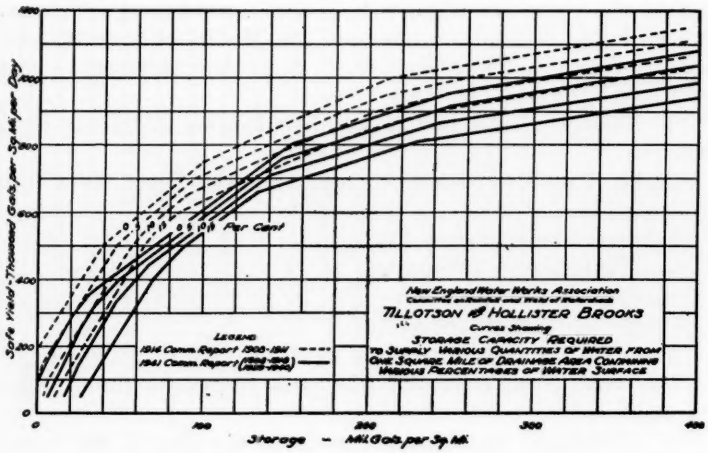

Safe Yield - Thousand Gals. per Sq. Mi. per Day
0
100
200
300
400
Storage - Mil Gals. per Sq. Mi.
0 5 10 15 20 Per Cent
LEGEND
1916 Comm. Report 1905-1911
1941 Comm. Report (1928-1932)
New England Water Works Association
Committee on Rainfall and Yield of Watersheds
TILLOTSON & HOLLISTER BROOKS
Curves Showing
STORAGE CAPACITY REQUIRED
TO SUPPLY VARIOUS QUANTITIES OF WATER FROM
ONE SQUARE MILE OF DRAINAGE AREA CONTAINING
VARIOUS PERCENTAGES OF WATER SURFACE

FIGURE 13

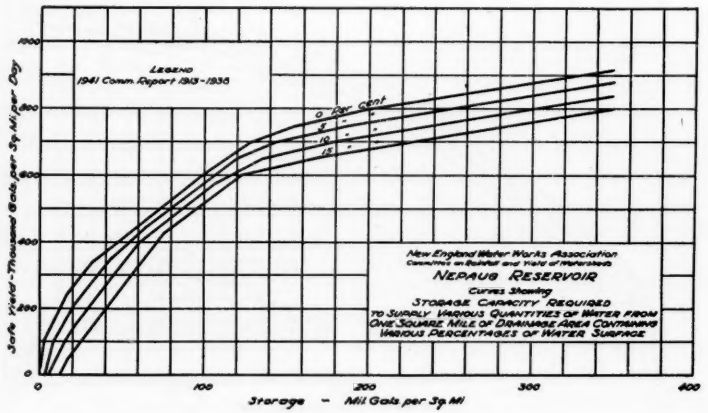


FIGURE 14

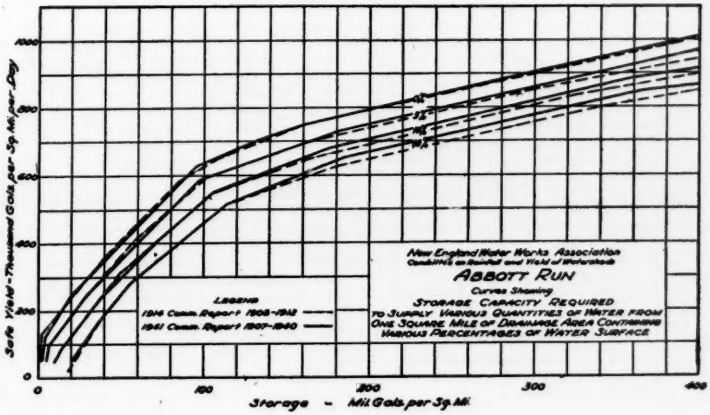


FIGURE 15

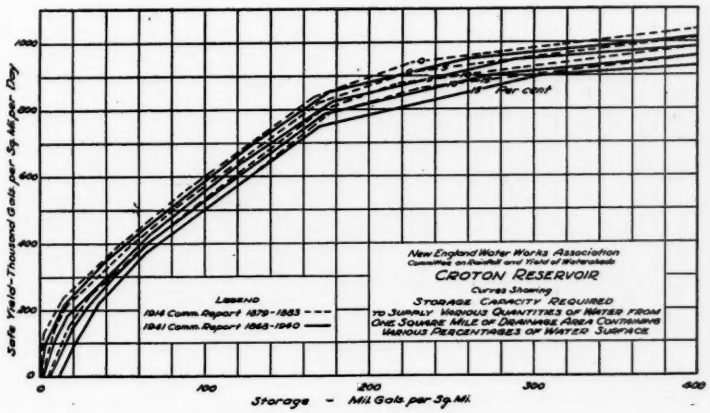


FIGURE 16

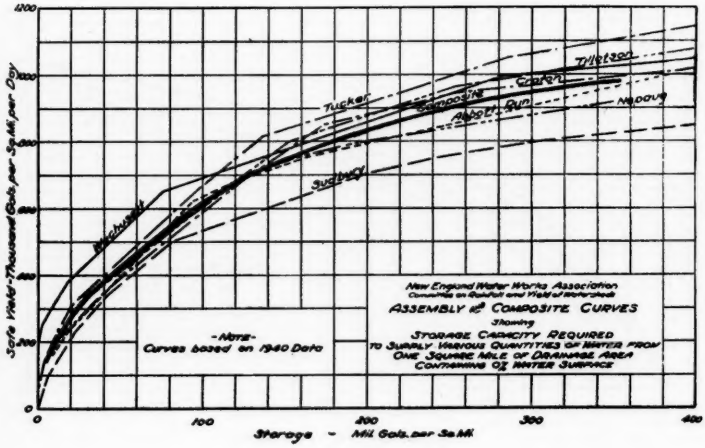


FIGURE 17

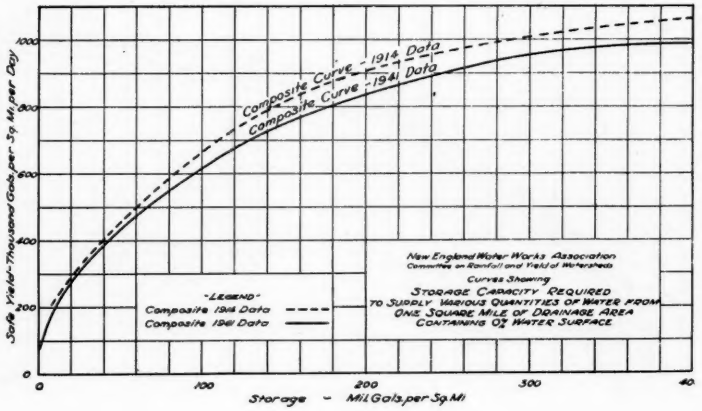

Safe Yield-Thousand Gals. per Sq. Mi. per Day
Storage - Mil Gals. per Sq. Mi
Composite Curve - 1914 Data
Composite Curve - 1941 Data
-LEGEND-
Composite 1914 Data
Composite 1941 Data
New England Water Works Association
Committee on Rainfall and Yield of Watersheds
Curves Showing
STORAGE CAPACITY REQUIRED
TO SUPPLY VARIOUS QUANTITIES OF WATER FROM
ONE SQUARE MILE OF DRAINAGE AREA
CONTAINING 0% WATER SURFACE

FIGURE 18

2. Ipswich River at South Middleton.
3. Ipswich River at Russell's Bridge.
4. Neponset River at Pleasant Street, Norwood.
5. Manhan River and Tucker Brook in Southampton.
6. Tillotson's Brook in Granville.

These streams drain three types of watersheds: (1) comparatively low and flat with few dams but with large swampy areas, such as the Charles and Ipswich sheds; (2) highly regulated, by dams and reservoirs, as is the Neponset; and (3) the natural flow from high, rough, mountainous, wooded areas as is the case of the Manhan and Tillotson sheds.

Charles River at Charles River Village, Mass. This gaging station was established by the Department of Public Health in 1913, and is located about 1000 ft. below the old dam at Charles River Village. Through July 1916, the records are from two staff gage readings daily; in August 1917 a recorder was installed from which the records have since been taken. In October 1937, the station was turned over to the U. S. Geological Survey.

This location was the only place on the River where the drainage area was 100 or more square miles in size and satisfactory low flows could be obtained. It was established primarily for low flows. By extending the fairly well defined rating curve above 400 cfs with many current meter check measurements of high flows, it has been possible to tabulate results, that are considered quite accurate.

Table 6 shows the measured flow at this station expressed in cubic feet per second per square mile by months, the maximum and minimum monthly flows, and the median for the period from October 1913 through September 1940.

The drainage area above this station is 184 sq. miles.

Ipswich River at South Middleton, Mass. This station was established in 1911 by the Department of Public Health at an old farm road bridge crossing the river about one mile below the old Paper Mill Dam at South Middleton. It was primarily a low-flow station to furnish information necessarily required by the diversion of flow of the river by the city of Lynn and later the city of Peabody. By extending the rating curve above 250 cfs the higher flows in the river are considered probably fair.

TABLE 6.—DISCHARGE OF THE CHARLES RIVER AT CHARLES RIVER VILLAGE.
Flow in cfs per sq. mile

	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.
1913-14	.803	.891	1.329	1.453	2.050	2.610	3.260	2.613	.572	.523	.421	.283
1914	.248	.330	.432	2.598	3.028	1.492	1.312	.893	.317	1.816	1.326	.354
1915	.664	.614	1.150	1.399	1.648	3.192	3.754	2.400	2.144	1.705	.885	.540
1916	.282	.277	.447	.823	.695	2.729	2.404	1.967	1.991	.600	.537	.706
1917	.914	1.349	1.001	1.037	2.435	3.146	2.135	1.501	.644	.521	.316	.856
1918	.744	.641	1.261	1.803	1.281	3.163	2.516	2.076	.791	.506	.331	1.583
1919	.431	1.973	2.067	.727	1.507	4.911	3.822	2.814	2.511	.987	.509	.353
1920	.266	.831	1.690	1.555	1.150	2.971	1.824	2.604	.659	2.451	1.187	.354
1921	.291	.696	1.898	.628	1.227	3.215	3.082	2.442	2.554	2.601	1.185	2.398
1922	1.012	.736	.580	2.760	2.373	3.591	3.297	2.148	.902	.764	.542	.263
1923	.552	.781	2.683	2.548	1.230	2.510	3.795	2.332	.927	.353	.436	.967
1924	.379	.333	.466	.359	2.602	2.585	2.089	.910	.610	.627	.525	.577
1925	.532	.987	2.372	1.298	1.604	3.489	2.638	1.337	.585	.387	.842	.350
1926	.470	1.134	.970	2.046	2.263	2.897	1.215	1.112	.726	.633	2.209	2.705
1927	1.894	2.919	3.431	1.982	2.416	1.929	1.883	2.220	2.190	2.084	1.445	1.440
1928	1.035	.928	1.021	2.090	1.769	3.355	3.122	2.784	1.070	.727	.349	.277
1929	.346	.433	.656	1.284	1.509	2.022	1.606	.647	.559	.477	.340	.213
1930	.189	.901	.609	1.082	1.409	3.502	2.689	2.060	2.902	1.104	.710	.394
1931	.218	.348	.877	1.437	1.346	2.259	2.613	.981	.690	.520	.373	1.190
1932	1.705	3.310	1.450	1.704	1.766	3.847	4.374	1.725	.842	.599	.360	1.541
1933	.952	.706	1.094	2.685	1.227	4.502	4.650	1.986	1.026	.300	.181	.448
1934	.930	.841	1.341	2.757	2.964	3.695	3.494	1.736	1.862	.528	.285	.497
1935	.266	.515	.641	1.933	1.325	6.246	3.509	2.293	.732	.483	.379	1.156
1936	.920	.683	3.265	3.114	2.511	2.429	2.484	2.299	1.766	1.299	1.011	1.380
1937	.620	1.375	3.033	2.60*	2.97	2.28	1.99	1.57	1.84	5.78	3.34	1.99
1938	1.59	1.51	2.51	1.40	2.25	3.80	4.60	1.65	.777	.367	.378	.260
1939	.303	1.070	.788	.984	.902	3.260	5.580	2.280	1.430	.848	.292	.229
1940	.212	1.023	1.077	1.201	2.532	1.636	1.737	1.097	.818	—	—	—
Max.	1.894	3.310	3.431	3.114	3.028	6.246	5.580	2.814	2.902	5.780	3.340	2.705
Min.	.189	.277	.432	.359	.695	1.492	1.215	.647	.317	.300	.181	.213
Med.	.552	.831	1.150	1.555	1.648	3.163	2.689	2.060	.902	.627	.509	.540

The third decimal is not considered accurate at high flows.
*From January 1938 corrections made for diversions.

Readings were made daily from a staff gage, but owing to the illness of the observer the daily record had to be discontinued in 1934. This station was turned over to the U.S.G.S. in 1938, who changed the location to a point on the river about 1000 ft. below the Paper Mill Dam and built an artificial control and a concrete gage house equipped with a recorder.

Table 7 shows the measured flow at this station expressed in cubic feet per second per square mile by months, the maximum and minimum monthly flows, and the median for the periods from October 1911 to September 1933 and from June 1939 to September 1940.

The drainage area above the original location was 44.57 sq. miles. At the new location, the drainage area is 43.4 sq. miles.

Neponset River at Norwood, Mass. This gaging station is located just above the Pleasant Street Bridge near the Morrill Ink Works.

The record was started on October 23, 1917, at which time a recorder was installed. It was maintained quite continuously by the Department of Public Health through October, 1939, when the station was turned over to the U.S.G.S. Owing to the large amount of manufacturing wastes and sewage discharged into this stream, the establishing of a rating curve required much care. The results are considered good except for high stages with the river out of its banks or above 210 cfs.

All except the high flows are subject to regulation in mill ponds and reservoirs.

Table 8 shows the average monthly measured flow at this station expressed in cubic feet per second per square mile, the maximum and minimum monthly flows, and the median for the period from November 1917 to September 1938.

The drainage area above this station is 35.6 sq. miles.

Manhan River and Tucker Brook at Southampton, Mass. Measurements of flow of the Manhan River and Tucker Brook, a tributary, were started during the summer of 1896 when the standard weirs with sharp crests were built across each stream. The combined drainage area above these weirs was 13.0 square miles (7.25 sq. mi. on the Manhan River and 5.75 sq. mi. on Tucker Brook). Table 9 shows the combined flow of both streams for the period from January 1897

.540 .509 .627 .902 2.060 2.689 3.163 1.048

The third decimal is not considered accurate at high flows.
*From January 1938 corrections made for diversions.

TABLE 7.—DISCHARGE OF THE IPSWICH RIVER AT RUSSELLS BRIDGE.

Flow in cfs per sq. mile

	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.
1911-12	.647	1.174	1.810	1.531	1.815	4.042	4.053	2.514	1.043	0.376	.367	.271
1912	.265	.591	.721	2.432	1.139	2.987	3.335	1.508	.761	.037	.032	.220
1913	1.156	1.253	1.638	1.126	2.017	4.350	4.325	3.317	0.526	.412	.181	.077
1914	.166	.173	.308	1.737	2.508	1.397	1.241	1.030	.373	3.481	3.021	.569
1915	.594	.760	1.689	2.314	2.418	3.696	4.551	3.637	3.403	1.097	.568	.343
1916	.178	.232	.535	.794	1.213	3.981	3.095	2.143	1.973	.370	.179	.147
1917	.548	.829	.662	.515	2.528	3.771	2.570	1.023	.438	.426	.223	.941
1918	1.081	.814	1.275	1.696	1.003	4.308	2.770	2.596	.819	.238	.266	1.233
1919	.577	2.358	2.186	.628	1.174	*	*	3.301	2.774	.699	.388	.435
1920	.438	1.464	2.813	2.138	1.207	3.884	2.434	2.794	.508	3.653	1.300	.258
1921	.300	.961	3.178	.967	1.383	4.597	3.743	3.008	4.554	3.234	.815	1.083
1922	.853	.614	.498	2.873	2.986	5.123	5.331	2.804	.616	.214	.175	.034
1923	.215	.621	2.550	3.101	1.334	3.607	4.356	2.743	.786	.172	.133	1.831
1924	.599	.503	.747	.320	3.442	4.039	3.919	1.205	.472	.391	.196	.048
1925	.456	1.174	3.305	1.250	1.897	4.855	4.222	1.586	.613	.135	.180	.046
1926	.227	.873	.772	1.735	1.460	3.088	0.612	0.523	0.270	.165	.364	.659
1927	.877	1.540	2.961	1.544	2.110	2.900	2.516	2.490	2.317	1.704	0.615	0.610
1928	.749	.834	1.091	1.490	1.754	3.959	3.921	2.865	.350	.113	.105	.138
1929	.227	.155	.274	1.066	1.369	2.152	1.506	.803	.478	.181	.144	.097
1930	.179	.675	.489	.819	1.451	4.298	3.341	1.775	2.601	.941	.325	.240
1931	.234	.330	.692	1.716	1.362	2.666	2.849	.868	.251	.153	.243	.484
1932	2.000	4.298	1.348	1.606	1.598	5.420	5.940	1.594	.463	.206	.155	1.748
1933	1.396	.803	.720	2.329	.974	4.591	4.537	1.227	.644	.201	.103	.198
1938	—	—	—	—	—	—	—	—	1.399	4.516	2.221	2.396
1938-39†	1.64	1.60	2.86	1.22	1.76	3.87	6.04	1.70	.839	.200	.085	.064
1939-40†	.157	.493	.537	.624	.700	3.27	6.52	2.47	1.58	.599	.108	.145
Max.	2.000	4.298	3.305	3.101	3.442	5.420	6.52	3.637	4.554	4.516	3.021	2.396
Min.	0.157	0.155	0.274	0.320	1.003	1.397	.612	.523	0.251	0.037	0.032	0.034
Med.	0.548	0.814	1.091	1.531	1.460	3.922	3.831	1.775	0.702	0.373	0.233	0.264

The third decimal is not considered accurate at high flows.
*Gage inaccessible, because of excessive high water.
†Provisional.

TABLE 8.—DISCHARGE OF THE NEPONSET RIVER AT NORWOOD.

Flow in cfs per sq. mile

	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.
1917-18	—	1.953	1.478	.986	1.592	2.548	2.796	2.134	1.085	.851	.600	1.727
1918	1.332	1.052	1.640	2.161	1.709	3.545	3.140	2.767	1.133	.925	1.091	2.682
1919	1.676	2.254	2.392	1.420	1.821	7.405	4.767	3.605	3.126	1.633	1.105	.675
1920	.558	1.346	2.081	2.000	1.767	3.442	2.877	3.628	1.126	3.463	1.817	.862
1921	1.411	2.006	2.550	1.705	2.135	3.595	3.761	3.300	4.063	2.804	2.177	2.918
1922	2.431	2.446	1.959	4.292	3.573	5.829	4.404	3.231	1.097	.670	.477	.366
1923	.575	.685	1.641	1.807	1.144	2.217	3.610	2.137	.827	.391	.602	1.521
1924	.694	.694	.717	.556	2.089	2.480	2.188	1.160	.723	.607	.461	.688
1925	.833	.975	2.076	1.268	2.032	3.208	2.663	1.436	.857	.439	.610	.474
1926	.655	1.262	1.342	2.294	2.139	2.474	1.059	1.026	.693	.437	1.218	1.854
1927	1.812	2.125	3.904	2.439	2.384	2.004	2.081	2.004	1.766	1.225	.605	.874
1928	1.403	1.077	1.011	2.035	1.795	3.244	3.140	2.500	1.075	.577	.362	.413
1929	.607	.808	.947	1.317	1.545	2.156	1.398	.814	.797	.311	.574	.333
1930	.569	.506	.809	.912	1.860	4.052	3.804	2.345	3.757	1.282	.662	.647
1931	.570	.454	.583	1.502	1.119	2.283	2.341	1.023	.627	.424	.566	.818
1932	1.466	3.194	1.506	1.337	1.575	4.243	5.131	1.464	.703	.546	.562	1.663
1933	1.519	.926	1.167	2.182	.676	3.330	3.323	1.650	.742	.300	.363	.327
1934	.685	.889	1.672	2.296	1.944	5.310	3.397	1.296	1.224	.634	.355	.328
1935	.339	.432	.428	1.306	.832	5.420	3.338	1.455	.639	.457	.407	.820
1936	.855	.618	3.670	3.224	2.090	2.229	2.677	2.005	1.537	.485	.328	.828
1937	.948	1.971	3.249	2.417	2.607	2.605	2.032	1.949	2.270	4.687	2.319	2.188
1938	2.021	2.006	2.747	1.877	3.224	4.443	4.638	1.720	.588	.472	—	—
Max.	2.431	3.194	3.904	4.292	3.573	7.405	5.131	3.628	4.063	4.687	2.319	2.918
Min.	.339	.432	.428	.556	.676	2.004	1.059	.814	.588	.300	.328	.327
Med.	.855	1.065	1.656	1.842	1.840	3.287	3.140	1.977	1.080	0.592	.600	.820

The third decimal is not considered accurate at high flows.

to December 1910 inclusive. During these fourteen years the total runoff of Tucker Brook exceeded that of Manhan River by only 1.72 per cent. Beginning with January 1911, the measured flow is from Tucker Brook only (5.75 sq. miles).

Table 9 shows the average monthly measured flow at these stations expressed in cubic feet per second per square mile, the maximum and minimum monthly flows, and the median for the period from January 1897 to December 1939.

These measurements were carried on by the Water Department of the city of Holyoke of which P. J. Lucey is Engineer. We are indebted to him for the basic data.

Tillotson Brook at Granville, Mass. Measurements of the flow of this stream were started in January 1908 by the building of a weir across the stream at what is now the dam site of the Granville Reservoir. The drainage area above this weir was 5.09 square miles (adjusted from 5.84 sq. mi.), heavily forested and mountainous. There was no regulation.

Flow measurements are missing from July 1918 to August 1928. During this time, the Granville Dam and Reservoir were completed and two weirs just above the reservoir were built, one across Tillotson Brook, the other on the tributary known as Hollister Brook, the combined area of these adjacent watersheds being 4.35 square miles.

Table 10 shows the average monthly measured flow at these stations expressed in cubic feet per second per square mile, the maximum and minimum monthly, and the median for the periods from January 1908 to July 1918 and from August 1928 to September 1940, except for the missing months of January, February, March, and April, 1929. The combined flows of both streams are shown in the latter portion of the table.

These measurements were carried on by the Public Works Department of the City of Westfield, Oron E. Parks, Superintendent, to whom we are indebted for the basic data.

TABLE 9.—DISCHARGE OF THE MANHAN RIVER—HOLYOKE WATER WORKS.

Flow in cfs per sq. mile

Drainage area 13.0 sq. miles

	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.
1897	—	—	—	.998	1.657	3.775	3.229	1.807	2.521	7.056	2.957	.902
1897-98	.491	1.736	3.560	2.132	1.728	5.843	3.548	3.779	1.913	.534	1.037	.461
1898	1.965	2.960	2.350	2.275	1.119	2.527	7.622	1.278	.384	.565	.198	.461
1899	.306	.672	.698	.730	6.421	4.334	3.071	2.353	.579	.231	.263	.122
1900	.283	1.537	1.472	.470	.305	3.426	10.084	6.141	1.572	.447	.969	1.335
1901	1.187	.812	4.090	3.624	2.005	6.638	3.598	1.540	.613	.863	.777	1.181
1902	2.556	1.414	2.808	2.279	3.214	6.274	4.260	0.849	4.207	1.762	2.278	1.004
1903	2.101	1.495	2.417	1.094	1.024	4.100	7.351	2.694	3.279	.993	.778	1.312
1904	1.448	.955	.989	3.508	.910	4.565	3.980	.979	.585	.729	.359	2.794
1905	1.032	.961	1.499	1.778	1.133	2.527	4.648	2.145	.966	.795	.483	.289
1906	1.052	1.142	1.157	2.016	.846	3.601	2.290	1.906	1.510	.464	1.470	2.375
1907	3.477	4.851	2.491	2.751	2.264	3.831	3.010	4.401	1.140	.650	.531	.263
1908	.368	.391	.426	1.047	3.206	3.700	5.898	2.412	1.139	.195	.444	.472
1909	.277	.299	.659	3.752	1.374	5.705	4.473	2.162	1.849	.207	.388	.300
1910	.203	.699	.405	—	—	—	—	—	—	—	—	—

TUCKER BROOK

Drainage area 5.75 sq. miles

1911	—	—	—	.585	.371	1.998	2.674	.986	1.171	.702	.579	.593
1911-12	3.257	2.262	1.745	1.128	1.080	4.119	4.710	2.680	.713	.234	.337	.326
1912	1.120	1.574	1.739	2.683	1.393	4.681	3.288	2.035	.528	.170	.101	.314
1913	1.140	1.029	1.445	.817	1.290	3.693	5.745	3.104	.483	.450	.249	.113
1914	.183	.370	.456	2.453	3.420	1.735	2.709	1.451	.543	3.265	4.510	.546
1915	.478	2.244	2.444	2.440	4.500	2.442	6.069	2.507	2.397	1.037	.484	.764
1916	.582	1.253	1.433	1.755	.885	3.274	3.680	3.003	2.431	.658	.313	.179
1917	1.509	.873	.741	.715	1.388	5.024	3.735	1.934	.860	.422	.402	.803
1918	.426	.806	1.552	1.481	.778	4.415	3.088	3.899	1.216	.543	.330	.698

TABLE 9.—(*Continued*)

	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.
1919	.514	2.007	1.557	.775	.564	6.518	7.552	3.006	2.593	.819	.881	1.194
1920	1.287	2.280	5.103	1.755	1.152	5.262	4.034	2.370	.734	1.442	.399	.215
1921	.356	.807	1.186	.351	.736	.508	4.347	4.681	3.709	1.528	1.907	1.327
1922	.734	.719	.653	1.741	.826	3.288	7.214	2.502	1.153	.326	.237	.378
1923	1.388	1.224	3.505	3.212	.859	1.832	8.708	2.972	.877	.209	.285	1.015
1924	.538	.992	.966	.384	3.955	3.811	2.386	1.557	.914	1.464	.630	.347
1925	.882	2.078	2.562	1.349	.939	2.001	4.715	1.623	.760	.509	.981	.367
1926	.863	2.313	1.236	2.180	2.435	4.857	1.803	2.254	1.865	.877	1.505	1.061
1927	2.750	6.791	4.061	2.253	3.273	2.128	3.415	3.036	3.412	2.157	2.126	2.613
1928	.730	.678	.675	1.431	1.256	3.092	5.516	2.788	1.445	.254	.147	.119
1929	1.075	.617	.970	1.151	1.346	2.538	2.470	1.569	2.593	.613	.231	.167
1930	.094	.933	.361	.485	.402	2.001	4.199	5.069	3.173	1.466	.631	.303
1931	.330	.306	.555	2.698	1.780	1.407	4.099	1.253	.943	.606	.227	.241
1932	1.481	4.868	1.554	1.605	1.465	2.374	6.476	1.649	1.167	.342	.774	3.661
1933	1.296	1.130	2.486	3.336	1.136	3.472	7.212	2.865	1.931	.351	.463	3.181
1934	1.295	2.062	1.809	3.217	1.487	3.208	2.308	1.510	1.494	.671	.170	.572
1935	.156	.791	.748	1.160	.518	15.262	3.885	1.348	.476	.153	.202	.588
1936	.811	.518	2.495	3.297	3.256	2.446	4.146	4.652	1.520	1.495	.895	1.214
1937	3.061	4.408	2.434	4.230	2.505	2.057	2.424	1.873	2.304	3.382	1.322	11.305
1938	1.804	1.576	3.182	1.567	1.867	3.528	5.266	1.488	0.876	0.420	0.734	0.361
1939	.406	.883	.707	—	—	—	—	—	—	—	—	—
Max.	3.477	6.791	5.103	4.230	6.421	15.262	10.084	6.141	4.207	7.056	4.510	11.305
Min.	0.094	0.299	0.361	0.351	0.305	0.508	1.803	0.849	0.384	0.153	0.101	0.113
Med.	0.882	1.130	1.552	1.755	1.346	3.528	4.099	2.254	1.216	0.613	0.484	0.588

TABLE 10.—DISCHARGE OF TILLOTSON BROOK—WESTFIELD WATER WORKS.

TABLE 10.—DISCHARGE OF TILLOTSON BROOK—WESTFIELD WATER WORKS.

Flow in cfs per sq. mile

Drainage area 5.09 sq. miles

	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.
1908	—	—	—	3.682	2.765	4.705	2.888	3.435	1.323	.788	.777	.339
1908-09	.499	.430	.657	1.281	3.013	3.994	4.899	2.818	1.196	.398	.825	.448
1909	.380	.492	.806	3.033	2.297	5.687	3.238	1.900	1.853	.374	.327	.393
1910	.272	1.035	1.162	.689	.694	2.546	2.962	.817	1.019	.272	.644	.896
1911	3.291	1.871	2.498	1.654	1.536	4.848	4.463	3.298	.917	.663	.447	.618
1912	2.091	1.077	1.612	2.605	1.814	3.677	3.415	2.035	1.004	.280	.260	.524
1913	2.741	1.963	1.928	1.383	1.968	4.341	5.226	3.494	.930	.620	.298	.196
1914	.314	.748	.566	2.940	3.851	1.701	2.353	1.493	.812	1.368	2.386	.641
1915	.764	1.196	1.598	3.030	2.671	3.530	6.493	2.905	3.570	1.577	.562	1.220
1916	.553	.730	1.429	1.667	1.384	4.865	5.529	2.514	4.047	1.129	1.041	.877
1917	1.501	2.607	1.157	1.644	2.513	5.317	4.188	1.883	.895	.464	—	—
					TILLOTSON & HOLLISTER BROOKS							
					Drainage area 4.35 sq. miles							
1928	—	—	—	—	—	—	—	—	—	—	2.746	1.715
1928-29	.735	.673	.868	—	—	—	—	3.242	.573	.187	.099	.125
1929	.312	.509	.851	1.240	1.153	2.881	2.748	1.846	2.012	.769	.228	.150
1930	.147	.855	.416	.468	.642	2.224	4.571	3.614	3.829	.709	1.049	.169
1931	.178	.246	.490	1.595	1.263	1.373	4.625	1.318	.918	.456	.239	.144
1932	.851	5.573	1.625	1.602	1.722	3.408	6.702	1.641	.494	.163	.461	1.535
1933	.666	.740	.883	1.677	.820	3.203	6.967	2.956	1.180	.325	.285	4.214
1934	1.605	2.046	2.275	2.614	1.494	3.320	2.337	1.547	1.519	.380	.115	.221
1935	1.544	.475	.384	1.062	.570	9.508	4.182	1.370	.412	.123	.157	.209
1936	.575	.507	2.933	3.810	3.491	2.318	4.156	3.370	1.328	.976	.252	.431
1937	.790	2.550	2.523	4.699	2.441	2.209	2.483	2.638	2.364	2.174	1.303	3.405
1938	1.431	1.280	3.675	1.599	1.531	3.286	4.892	1.276	.355	.162	.663	.127
1939	.211	.430	.425	.612	.383	1.334	6.736	3.211	1.915	.562	.172	.133
Max.	3.291	5.573	3.675	4.699	3.851	9.508	6.967	3.494	3.829	2.174	2.746	3.405
Min.	0.147	0.246	0.384	0.468	0.383	1.334	2.353	0.817	0.355	0.123	0.099	0.125
Med.	0.700	0.802	1.160	1.648	1.629	3.364	4.544	2.514	1.196	0.464	0.447	0.431

For the Committee,

(Signed) ARTHUR D. WESTON, *Chairman*

PROCEEDINGS

May 1945 Meeting

Hotel Statler, Boston, Mass.

Thursday, May 17, 1945

President Karl R. Kennison in the chair.

Acting Secretary Shaw announced that the following had been elected to membership by the Executive Committee:

Members: Otto J. Meotti, Superintendent of Water Department, Woonsocket, R. I.; Ernest D. Mortenson, Chief Engineer and Superintendent of Water Works and Sewage, U. S. Naval Advance Base Depot, Water Works Department, Davisville, R. I.; Robert W. Moir, with Metropolitan District Water Supply Commission, Auburndale, Mass.; Harold John Toole, Director, Water Division, Metropolitan District Commission, Clinton, Mass.

Corporate: Bellingham Water Board, Bellingham, Mass.

A paper, "Military Use of Public Water Supplies," was read by Lieutenant-Colonel Robert N. Clark, Sanitary Corps, First Service Command, U. S. Army, Boston, Mass.

A paper, "Chlorine Dioxide Treatment of Water Supplies," was read by Joseph A. McCarthy, Chief of Laboratory, Lawrence Experiment Station, Massachusetts Department of Public Health, Lawrence, Mass.

A paper, "New Developments in Electronic Instrumentation," was read by D. M. Neilson of The Foxboro Company, Foxboro, Mass.